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# RECOMMENDED PRACTICE FOR THE DESIGN AND CONSTRUCTION OF EARTH DAMS FOR INDUSTRIAL AND POTABLE WATER SUPPLY IN THE FAR NORTH AND PERMAFROST AREAS

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L.N. Kuz'mina, ed.

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These recommendations examine the problems of erecting permanent earth dams in water-engineering systems built to supply potable and process water. The dams in question are frozen dams, intended for operation with the impervious element and its foundation kept permanently in the frozen state. The recommendations also examine specific problems of erecting thawed dams in the Far North.

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Preface

These Recommendations examine the problems of erecting permanent earth dams in water-engineering systems built to supply potable and process water. The dams in question are frozen dams, intended for operation with the impervious element and its foundation kept permanently in the frozen state. The Recommendations also examine specific problems of erecting thawed dams in the Far North.

Not considered are large water-engineering systems used for hydroelectric power, and temporary or periodically-operating dams (embankments) used for lagoon irrigation or flooding of dredging ponds or temporary water intakes.

The Recommendations were worked out by the Laboratory of Hydraulic-Engineering Structures at VODGEO;* team members were V. I. Titova and Yu. I. Svateyev, Candidates of Applied Sciences; I. S. Kleyn and M. S. Rodovanskaya, Engineers; and L. M. Zhurkova, Senior Technician. Also participating in the work was the Permafrost Geotechnical Laboratory at the Krasnoyarsk Polytechnic Institute (G. I. Kuznetsov, Candidate of Applied Sciences).

The Recommendations have been compiled with reference to experience gained in the planning, construction and operation of dams in the Far North. Among the planning, construction, and scientific-research organizations that have gathered this experience are Gidroproyekt and its Leningrad Branch, Dal'stroyproyekt, Yakutniproalmaz, Vilyuygesstroy, Khantaygesstroy, the Noril'sk Combine, VNIIG and its Siberian Branch, the Institute of Cryopedology of the USSR Academy of Sciences, Siberian Branch; the Gorky and Moscow Institutes of Engineering and Construction; the Northeast Combined Scientific Research Institute of the USSR Academy of Sciences; and the Promstroyniiproyekt organization at Krasnoyarsk.** The results of six All-Union conferences and meetings

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*All-Union Scientific Research Institute of Water Supply, Sewer Systems, Hydraulic Engineering Structures, and Engineering Hydrology.

**The abbreviations and acronyms denote: Gidroproyekt = S. Ya. Zhuk All-Union Planning, Surveying and Scientific Research Institute. Dal'stroyproyekt = construction and planning administration for the Soviet Far East. Yakutniproalmaz = a scientific research and planning institute for the diamond industry of the Yakut ASSR. Vilyuygesstroy and Khantaygesstroy = organizations for the construction of the Vilyuysk and Khantaysk hydroelectric power plants. VNIIG = B. E. Vedeneyev All-Union Scientific Research Institute of Hydraulic Engineering. Promstroyniiproyekt = a scientific research and planning institute for industrial construction.

on hydraulic engineering in the Far North and of the Second International Conference on Cryopedology (Yakutsk, 1973) were also taken into account.

Many of the Recommendations are based on advances in engineering cryopedology and in the mechanics of frozen soils that have been reported by N. A. Tsytovich, Corresponding Member of the USSR Academy of Sciences.

The compilers of the Recommendations have used the results of full-scale observations on water-supply dams, both under construction and in operation, in various regions of the North--on the Chukchi Peninsula, in the Magadan Oblast and the Yakut ASSR, and near the city of Noril'sk--and also on the large dams at the Vilyuysk and Khantaysk hydroelectric power plants.

The compilers have also used the results of work by leading specialists in dam construction in the North: G. F. Biyanov, Prof. P. A. Bogoslovskiy, L. E. Vedernikov, V. G. Gol'dtman, V. N. Grandilevskiy, V. V. Znamenskiy, R. M. Kamenskiy, G. I. Konenkov, A. L. Kuznetsov, N. N. Petrunichev, B. S. Suvorov, S. G. Tsvetkova, G. Ya. Yappu, the late G. S. Shadrin, and others.

The Recommendations also make use of research by S. V. Tomirdiaro on treatment of the banks of northern lakes and reservoirs; by Ya. A. Kronik on frost heaving; by S. E. Grechishchev on frost splitting; and by Yu. N. Myznikov on the working of soils and their storage and placement in dams.

The Recommendations on calculating the thermophysical characteristics of soils were made up from materials of L. T. Roman.

The compilers have gratefully received and heeded the comments and wishes of the following experts: I. A. Vasil'yeva (VNIIGiM); S. E. Grechishchev (Vsegingeo); P. I. Dulints (Promenergoproyekt, Irkutsk); the late Prof. K. A. Mikhailov, S. I. Migin and A. I. Pilyugin (VNII VODGEO); V. A. Sorokin (Dal'stroyproyekt, Magadan); V. S. Timofeychuk (VNIIProzoloto); S. V. Tomirdiaro (SVKNII SO AN SSSR, Magadan); and S. V. Bortkevich and N. A. Krasil'nikova (the NIS of Gidroproyekt).*

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*The new abbreviations and acronyms denote: VNIIGiM = A . N. Kostyakov All-Union Scientific Research Institute of Hydraulic Engineering and Reclamation. Vsegingeo = All-Union Scientific Research Institute of Hydrogeology and Engineering Geology. Promenergoproyekt = State All-Union Planning Institute for the Planning of Construction of Industrial Heat and Electric Power Plants for Supplying Power to Industrial Establishments of All Branches of the National Economy. VNII VODGEO = VODGEO. VNIIProzoloto = an All-Union scientific research and planning institute for the gold industry. SVKNII SO AN SSSR = northeast design and scientific research institute, USSR Academy of Sciences, Siberian Branch. NIS = scientific research station.

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1. General provisions

1.1. These Recommendations are for use in the planning and construction of earth and earth-rock dams with heads up to 25 m as components of water-engineering systems for process and potable water supply in regions of the Far North.

1.2. In the planning of dams for the north, the requirements of the following Chapters of the "Construction Standards and Regulations" should be taken into account:

Engineering surveys for construction: Basic provisions.

Hydrotechnical structures on rivers: Basic design provisions.

Dams from earth materials.

Earth structures.

1.3. Engineering-geological surveys and geotechnical inspection should be carried out in accordance with the following documents:

(a) Recommendations on the method of studying ground ice and the cryogenic structure of permafrosts. Moscow: PNIIIS,* 1969.

(b) Recommendations on the method of studying solifluction processes in engineering surveys. Moscow: PNIIIS, 1969.

(c) Recommendations on the method of studying thermokarst processes in engineering surveys in permafrost regions. Moscow: PNIIIS, 1969.

(d) Handbook on the determination of physical, thermophysical and mechanical characteristics of frozen soil. Moscow: Stroyizdat,** 1973.

1.4. These Recommendations examine the design and engineering features and calculations of earth and rock dams that are due to climatic, hydrogeological and cryological features of Far North regions, which regions are characterized by a mean annual air temperature below freezing and by permafrosts in the foundations.

1.5. The following features characterize the regions over which permafrost extends: long, hard winters; great variety of permafrost phenomena (including saturation of the ground with textured ice, often with large ice inclusions; thermokarst; icings; frost clefts and

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*Industrial and Scientific Research Institute for Engineering Surveys in Construction.

**State Publishing House of Construction Literature.

frost heaving; and solifluction, thaw settlement and thermal abrasion); wide depth range of seasonal thawing (from 0.3-0.5 m, when there is a cover of moss and vegetation, up to 3 m and more, when the ground is not covered by snow or vegetation); high wind speeds combined with large quantities of snow; and, in coastal regions of the Far North, frequent shifts in seasonal conditions.

1.6. The hydrological regimes of rivers and creeks utilized when reservoirs are created in the Far North have the following features:

Supply mainly by surface runoff, with violent floods.

Freezing of rivers and creeks to the bottom, often with freezing of the talik under the stream bed.

Considerable losses of reservoir volume to ice formation in the period when water is in shortest supply (the ice thickness reaches 2 m).

1.7. The creation of reservoirs greatly changes the natural conditions of the frozen soil in the bottom of the reservoir, on its banks, and in the lower pool. These changes may be accompanied by violation of the stability of the reservoir banks as they thaw, melting of buried ice, floating of submerged sections of the vegetation layer or of peat, and so on.

1.8. Planning for earth dams should take into account that the prolonged action of temperatures far below freezing offers the following threats: heaving of the foundation soil in the abutments and in the dam foundation; formation of ice clefts in the dam and abutments where there is no vegetation cover; and formation of icings where water seeping through or past the structures issues on the surface.

2. Requirements for surveys

2.1. For dams in temperate regions, the surveys needed for exploration of the site, borrow pits, and rock deposits and for justification of the planned project should be carried out in complete accordance with the Chapters of the "Construction Standards and Regulations" cited in Paragraph 1.2.

2.2. Surveys in northern regions should give the following additional information:

Changing forms of the landscape: solifluction slopes and rates of slumping; changing outlines of the shores of thermokarst lakes, and directions and rates of their migration.

Regimes of groundwaters--infrapermafrost, interpermafrost and superpermafrost--and features of the formation of icings.

Division of the annual precipitation into rain and snow. Wind

roses for the hot and cold seasons. Duration and frequency of blizzards; air temperature and wind speed in blizzards. Length of below-freezing periods and wind speeds during these periods.

2.3. The following should be determined at the dam site and in the surrounding territory:

(a) Thickness of permafrost and the nature of its distribution. Temperature of foundation soils to a depth not less than twice the head. Depth of the zone of seasonal temperature fluctuations. Depths of seasonal thawing in various soils and under various conditions.

(b) For buried ice: maximum depth of occurrence; types (wedges, layers); dimensions and orientations of the largest bodies, in plan and in a section along the pressure front of the water-engineering system and in the abutments.

(c) Dimensions and outlines of the talik under the stream bed, and seasonal changes.

(d) Degree of jointing of bedrock and variation of jointing with depth. Character of bedding, dips and strikes of beds. Dimensions and directions of joints; composition of joint filler.

(e) Ice content and total moisture content [Paragraph 1.3 (d)] of the loose foundation rocks.

(f) Thickness of the vegetation layer and other plant remains, and nature of their distribution. Dimensions of incompletely decayed organic inclusions and nature of their distribution.

(g) Frost heaving and frost splitting in the foundation soils near the site of the planned dam and its abutments.

(h) Temperature and ice conditions in the watercourse; temperature and ice conditions and their seasonal variations for the water and bottoms of nearby lakes.

(i) The presence, operating features and thermal regimes of existing hydrotechnical and other structures in the area of the planned reservoir and in the zone to which its hydrothermal influence will extend.

2.4. Reports from borrow-pit surveys should include:

(a) For each type of earth material needed for the dam: amount of reserves, bed thickness, depth of occurrence and irregularity in bed height; variation in irregularity over the pit (in plan); and content of organics.

(b) Flooding of the pits; possibility of draining or diverting

surface waters.

(c) Depth of seasonal thawing of soil under the vegetation layer and after its removal. Seasonal frost penetration of thawed ground in the loosened state, under an exposed surface and under a surface covered with a natural layer of snow.

(d) Estimate of how the regimes of superpermafrost and interpermafrost waters may affect the working of soils in the pits. Possibility of diverting from the pit these waters and those released when the soils thaw.

(e) The presence and amount of reserves of thawed, fine-grained soils making up the bottoms of nearby lakes that would be suitable for working and placing in the dam. Possibility of dewatering these soils.

2.5. Surveys in the flooded area and on the banks of the future reservoir must estimate:

(a) The danger of speeding up solifluction slumping of the shores through the eroding action of the reservoir.

(b) The danger of thawing buried ice inclusions and thaw settlement of frozen soils in the foundations of existing structures in the zone to which the thermal action of the reservoir will extend.

(c) Settlement of the bottom of the future reservoir (through a comparison with measurements of the bank elevations and depths of neighboring lakes; see Appendix 6).

(d) The danger that the reservoir will thaw underground ice inclusions at divides in gentle valleys and that the reservoir will break through into neighboring basins.

2.6. For the working drawings, the following must also be determined at the dam site:

(a) Physical-mechanical and thermophysical characteristics of foundation soils in the frozen and transition states (angle of internal friction, cohesion, compressibility, specific heat, thermal conductivity and diffusivity, deformability and strength indices of rock).

(b) Shear characteristics of the thawed layer with respect to the frozen layer and of the bedrock with respect to joints and interbedded ice.

(c) Relative thaw settlements of foundation rocks, and changes in settlement with depth. Bearing capacities of soils in thawed and transition states.

2.7. For the working drawings of borrow pits, the surveys must provide the following additional information:

(a) Densities, grain-size distributions and mineralogic compositions of the soils.

(b) Physical-mechanical characteristics of soils with disturbed structures (angle of internal friction, angle of repose, cohesion and compression in frozen and transition states, coefficients of permeability in the thawed state and variations of these coefficients with depth of the pit and also in plan).

(c) Moisture and ice contents of soil in its natural state. Content of unfrozen moisture in cohesive soils at various temperatures below freezing. Abilities of soils to give up moisture on thawing. Limits of the temperature spectrum of the phase transitions of the soil moisture. Conditions of the onset of seepage, and possibility of its development, in frozen and thawing soils.

(d) Abilities of soils in the borrow pits to bear loads due to machinery used in the working of the soils, in the thawing (transition) and thawed states.

3. Experimental work and research

3.1. At the start of dam construction, experimental work should be performed at the construction site and in the borrow pits to refine the plans for working the soils, placing them in the dam, and storing them in the thawed state for the winter. This work is done by the construction crew to the instructions of the planning organization. The construction estimate includes the cost of this experimental work.

3.2. The following work should be carried out at the experimental faces of a borrow pit:

(a) Selection of a rational method for loosening the frozen soil (blasting, loosening by impact or with tractors) and determination of the thawing rate of the loosened frozen soil.

(b) Selection of a rational method for thawing various frozen soils (hydrothawing, sprinkling, thawing under a film; thawing by exhaust gases from turbine engines, by fuel nozzles, by electric heaters, by passage of an electric current through the soil, etc.; combination of these methods).

(c) For rock pits, determination of the optimum arrangement of boreholes and method of blasting to obtain the optimum particle-size distribution of the rock.

(d) Determination of the weathering of weak rocks after dumping in an exposed state and under a snow cover.

3.3. The following choices must be made by experiment, near the dam site:

(a) Rational method for stockpiling thawed soil and storing it in the thawed state for the whole construction period (optimum dimensions of piles, use of artificial and natural depressions, salting and heating of the upper layer of soil in the piles).

(b) Rational method for dewatering waterlogged soils (holding in piles for a certain time, under a tent, addition of reagents to speed up release of water, use of the electroosmotic effect).

3.4. The following should be done on experimental layers of the dam:

(a) Selection of optimum dimensions of the layer and of the face in the working of trenches, depending on the season.

(b) Checking of the effectiveness of the planned method of transporting and distributing soils over the layer, under various weather conditions.

(c) Checking of the effectiveness of the method planned for compacting the soil (vibrational, impact, by rollers, traffic, tractors), depending on local conditions and opportunities.

(d) Development of a process and equipment for placing and welding polymer films to form impervious elements (including an experimental dump of underlying and protective layers).

(e) Selection of the optimum method, and development of a process, for thawing and compacting the foundation before construction.

(f) Checking of the effectiveness of the planned methods for preserving permafrost in the foundations of frozen dams and for freezing taliks.

(g) Determination of the rate of freezing of soil placed in the dam; checking of the effectiveness of planned methods for removing ice, snow and water from the surface of compacted soil when placement is resumed after an interruption.

3.5. The experimental work must establish permissible values for air temperature, wind speed and precipitation rate when various types of work are to be done in the borrow pit, at soil-storage sites, and on experimental layers of the dam. Estimates must be made of the seasonal periods when these permissible values are reached.

3.6. All types of experimental work should be accompanied by regular checks on the temperature, moisture content and density of the soils.

3.7. On the basis of the experimental work, the most efficient schemes for utilization of machinery should be worked out, and equipment and

processes should be developed for preparing the reservoir bottom for flooding.

4. Selection of dam type and site.
Makeup of water-engineering system

4.1. The selection of the dam type and site and the makeup of the water-engineering system depend on the site conditions (from cryopedological, engineering and hydrogeological standpoints), the availability of local construction materials, the predicted thermal effect of the reservoir on the frozen soil, and other factors.

4.2. Earth and earth-rock dams are divided, according to the principle of construction and the embankment temperature regime, into frozen and thawed types.

In frozen dams, the central part should be frozen solid at the moment when the reservoir is filled. A thawed dam can withstand the head of water in the reservoir when the impervious elements are in a thawed state. Seepage is not permitted in frozen dams; thawed dams, along with dams using film-type impervious elements, are designed as filter dams.

4.3. Dams made of local materials, and also the spillways of low-head dams in water-supply systems, should be designed on one of two principles with respect to the foundation: keeping the foundation frozen (principle I) or allowing it to thaw under the impervious elements of the dam (principle II).

4.4. In construction on the first principle, it is necessary to provide ways of keeping the temperatures below freezing in the permafrost foundation of a frozen dam and spillway and in abutments, or to lower the temperature of the frozen foundation by additional cooling where it makes contact with the crucial central part of the dam cross-section.

It is then inevitable that the foundation thaws under the upstream shell. Deformation of the upstream shell by thaw settlement of the soils need not result in a loss of overall stability of the structure.

4.5. In construction on principle II, the following developments are permitted:

Rise in the below-freezing temperatures of the permafrost foundation soils.

Thawing of the foundation while the dam is in operation, under the action of heat from the reservoir and seepage.

4.6. According to (1) the engineering, cryological, and geological features of structure and (2) the sensitivity to changes in the natural temperature and moisture regime in the zone to which the thermal effect

of the reservoir extends, foundations can be divided into the following types:

(a) Strong rocky foundations, thawed or frozen (disruption of the primary cryogenic structure of the frozen rock and of natural cryogenic processes taking place in the rock has no great effect on the strength, deformation, and permeability properties of the thawing foundation).

(b) Non-settling, non-rocky foundations (non-rocky thawed soils within the bounds of the talik under the stream bed, which is either exposed or else enclosed--surrounded by dense frozen rocks, not ice-heaved and not deforming upon thawing).

(c) Settling icy-rocky foundations (frozen rocky foundations with much jointing, ice saturation, and reduced strength in the upper, weathered, zone; any change in the natural temperature regime or cryogenic structure of the rock is accompanied by thaw settlement and an increase in permeability upon thawing).

(d) Frozen, loose sediments with high ice contents and limited thickness (compared to the height of the dam), underlain by non-settling rocks that are water-resistant upon thawing.

(e) Frozen, loose sediments with high ice contents and considerable thickness (often with large inclusions of ground ice), with substantial thaw settlement combined with thermokarst processes in the zone to which the thermal effect of the reservoir extends; the strength and bearing capacity of the soils drop sharply when they thaw.

4.7. In all cases where the foundation remains sufficiently reliable after thawing, a thawed dam should be planned. This type of dam is simpler in operation and, unlike the frozen type, does not require expenditures for cooling equipment or its operation.

4.8. It is desirable to plan thawed dams for foundations of types (a) and (b) (Paragraph 4.6). On foundations of types (c), (d) and (e), the use of thawed dams requires a complex set of steps aimed at preliminary thawing and compaction, or at replacing the thaw-settling icy foundation layers.

4.9. When it is clearly undesirable, from a technical-economic standpoint, to thaw before construction an icy foundation that would lose its bearing capacity or become pervious upon thawing, then a frozen dam, on the first principle of construction (keeping the foundation frozen), should be planned.

4.10. A frozen dam should be planned on any type of frozen foundation, provided that (a) the joints in the frozen rock are completely filled with ice or an impervious soil, (b) the thickness of the talik under the stream bed is less than 10 m, and (c) there are no layers of free-

flowing, frozen, loose sediments not cemented by ice. Preliminary freezing and subsequent maintenance of the talik in the frozen state are mandatory for the stability of a frozen dam.

4.11. Frozen dams with frozen central parts can be erected without preliminary freezing of the talik, provided that the seepage rate in the talik does not exceed 0.1 m/hr. This condition means that gradual freezing of the dam and foundation with freezing equipment is possible after the dam is erected.

4.12. The dam site should be picked as near as possible to the water-supply objective, sufficiently far away from sources of pollution over the design period of operation of the reservoir, and with allowance for the prospect of economic development of the region.

4.13. In site selection, the following facts must be taken into account:

(a) On solifluction slopes, after their natural thermal condition has been disrupted, large masses of thawing soil flow progressively into the reservoir. This significantly cuts down reservoir volume and pollutes the water.

(b) Structures standing on the shores of the reservoir may be deformed by thermal-erosion reworking of the banks, thawing of buried ice, significant thaw settlement of the soil, and acceleration of solifluction processes after the creation of the reservoir.

4.14. A combination of construction principles I and II, or of thawed and frozen dam designs, at a single site is not recommended.

4.15. In the planning of thawed dams, there are no special requirements on the way in which the dam combines with other structures of the water-engineering system.

4.16. In the planning of a frozen dam, works carrying water must not intersect the embankment. Intakes, spillways and outlets must be placed outside the embankment. If spillway works are set alongside the dam, a mass of frozen soil not less than 20 m long should be left between the works and the dam to protect the dam from thawing beginning next to the spillway and upper pool.

4.17. If a frozen dam is used to impound a cooling pond, the hot water should be discharged into the tail portion of the reservoir, and the intake should be situated not closer than 200 m to the dam.

5. Requirements on dam designs

Frozen dams

*Specific features of frozen dams*

5.1. Non-filter frozen dams make possible the maximum use of cryological and climatic features of Far North regions, in particular the rather high bearing capacity of frozen soils and their water-resisting properties. Reservoirs can be operated without water loss by seepage; this is especially important when dams are built on soils that become highly pervious on thawing, in regions with low moduli of surface runoff, in particular on shallow watercourses that freeze over.

5.2. The basic design elements that define the strength and imperviousness of a frozen dam are artificially-frozen or natural impervious frozen soils with ice-filled pores. The zone of permanently frozen soil occupies the middle and downstream parts of the dam; the zone of permanently thawed soil lies in the upstream part. The impervious barrier and the thrust to take up the water pressure are provided by the frozen downstream portion and the core of the dam. It is undesirable to use impervious linings located in the thawed upstream shell or on the upstream slope, since the deformations of the thawed zone when the foundation of the upstream shell thaws and settles may cause dangerous deformations of the lining. The upstream shell should be considered thermal insulation, preventing the central frozen zone from thawing.

5.3. There are various methods for creating a frozen, impervious dam embankment:

(a) Artificial freezing of the central portion (core) and the talik under the stream bed, after erection of the dam to its full height and before filling of the reservoir, or during the first years of reservoir operation, with the help of forced cooling by an air-cooled (less often, brine-cooled) freezing system or thermopiles.

(b) Layer-by-layer freezing of the thawed soil, either soil placed when the air temperature is a little below freezing or that placed in the warm season but freezable in the winter.

(c) Combined method: layer-by-layer freezing of the dam embankment after preliminary artificial freezing of the taliks.

(d) Freezing of the downstream shell of the dam in the winter, using natural ventilation of the space under a shelter on the downstream slope; freezing of the core with a freezing system. The ventilation ducts should be closed in the summer.

(e) Natural freezing of a dam made of thawed soils, resulting from the below-freezing mean annual outdoor air temperature and the permafrost of the foundation, when the impervious element is sufficiently water-resistant.

Each of these methods for setting up a frozen embankment should call forth appropriate design features.

Currently, the most widely-used method is freezing of the central zones

of the dam during the winter, using air-cooled freezing systems. This is a rather productive method, and simple to set up and operate. Under the most severe conditions, continuous brine-cooled freezing systems are used. Autonomous, automatic steam-condensing coolers have recently gone through rapid development, but for hydrotechnical construction these devices need considerable study aimed at devising simple, reliable, and sufficiently productive units.

5.4. A continuous freezing system with forced circulation of a liquid coolant (brine) through pipes in the freezing columns is a rather reliable method of freezing a dam.

Plans should take into account that the system is complex to install and requires careful preventive inspection so that brine leaks will not occur. If brine entered the soil, it would become impossible to freeze the soil.

This system may find limited use when it is necessary to freeze pervious taliks, for speeding up the freezing of a dam, or for local thermal reinforcement of the frozen zone in case of accidental thawing in isolated portions of the pressure front.

The use of complex cooling units with liquid coolants should be permitted only when there is appropriate technical and economic justification.

The use of thermopiles is recommended only after an experimental field check on cold output and parameters of the frozen-soil cylinder (radius, temperature, freezing rate).

5.5. In the erection of a frozen dam with layer-by-layer natural freezing to temperatures near the calculated limit (steady-state temperatures) in the center of the cross-section, thawing of the upstream shell does not extend past a vertical through the waterline on the upstream slope of slightly pervious soil. This vertical can be taken as the tentative boundary between thawed and frozen zones of a frozen-dam cross-section, when the mean annual outdoor-air temperature does not go above -8°C and the water temperature is not above +4°C.

5.6. Talik that develops in a reservoir bed in prolonged operation (25 years or more) of the reservoir does not as a rule extend past the waterline, if leakage and thermokarst processes do not occur in the rocks making up the bank slopes.

5.7. The natural cooling of a frozen dam is intensified under the following conditions:

The downstream slope faces north.

Steps are taken in late winter and spring to retain the snow on the slopes and crest. Snow retention is most effective when the snow layer melts slowly toward the end of summer and the soil

under the snow does not have time to thaw.

The snow is removed from the freezing downstream slope and crest when the outdoor-air temperature goes and stays below freezing.

Efficient thermal insulators are used to make seasonal (spring and summer) coverings on the crest and downstream-slope surfaces. An example of such a material is quick-hardening foam of any composition and strength. No requirements are made on the lifetime or moisture-resistance (the covering should be removed before the onset of continual below-freezing temperatures).

Temporary or permanent coverings (shelters), ventilated throughout the period of continual below-freezing air temperatures, are installed. Control screens should be provided to cut off the flow of air in the warm season and during snowstorms. The design of the coverings should have thermal-insulation properties, in order to prevent summer heating of the air under the shelter and keep rainwater and meltwater from reaching the surfaces of the crest and downstream slope.

5.8. The following design features (Fig. 1) will make air-cooled freezing systems most efficient:

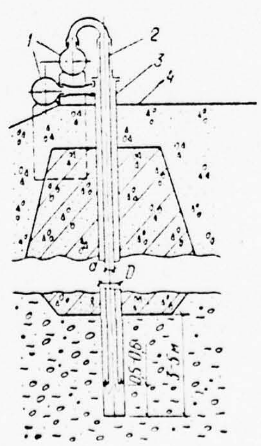


Fig. 1. Air-cooled freezing system. (*1*) Air lines; (*2*) inner pipe of column; (*3*) outer pipe of column; (*4*) crest of dam.

The inner pipe of the column should have a diameter half the inside diameter of the outer pipe. Small departures from this ratio are allowed only on the side of smaller diameter of the inner pipe:

$$\frac{D_{outer}}{d_{inner}} = 2\text{-}3.$$

The inner pipe should be suspended from a flange at the end of the outer pipe, with a clearance of not less than 50 cm above the bottom of the sealed outer pipe.

Air should enter and leave the column through inlet and outlet lines.

The inner pipe and the space between pipes should have flexible pipes hermetically connecting them to the air lines.

The intake and discharge openings of the air lines should have valves to cut off the flow of air into the system immediately after the blower is cut off.

For the preservation of the air-cooled freezing system through the warm season, the air lines and connecting pipes of inactive columns should be sealed by metal plugs seating on soft gaskets.

Each air system should be fitted with an automatic cutoff that acts when the outdoor-air temperature exceeds the surface temperature of the outer pipe of the column.

When the cooling system is under manual control, air should be supplied at temperatures below -10°C.

To reduce heating of the external parts of the air system by the sun's rays, the tops of the freezing columns and the air lines should be painted white, shaded by tents, or wrapped with heat-insulation tape.

Against the possibility that the system will have an insufficient cooling effect on isolated portions of a dam where the geological structure of the foundation is complex, there should be provisions for installing additional columns and connecting them to air lines during the operating period.

To compensate for the growth of rime on the freezing columns, the blower capacity should be not less than twice the value determined for normal service conditions.

In regions with high winter winds, the intake opening of the distributing air line should be provided with a lightweight filter and a three-dimensional labyrinth, the latter easily accessible

for regular removal of snow powder that has passed the filter.

The blower may be mounted either at the discharge opening of the outlet air line or at the intake opening of the inlet line. The choice of operating regime for the blower (blast or draft) is made by calculating the maximum cold productivity of the system.

If the foundation soils have the highest ice content, then cold outdoor air should be injected into the freezing column through the inner pipe and exhausted through the annular space between the outer and inner pipes. If the core of the dam has the highest ice content, then it is better to inject through the space between pipes and discharge through the inner pipe.

The outer pipe of the column should be made of steel or aluminum tubing, the inner one of plastic or plywood (to cut down heat exchange between the ascending and descending streams of air and to provide maximum cold transmission from the column to the ground). The method selected for heating the columns to eliminate ice plugs must be taken into account.

To reduce heating and thawing of the soil where it makes contact with summer-heated metal, a layer of heat insulation (slag or sawdust) should be placed around the freezing columns in the seasonally-thawing (active) layer of soil at the dam crest.

Each column should be fitted with a port for the introduction of instruments to measure the flow rate and air temperature.

5.9. The limiting temperature state of the frozen dam and reservoir bottom should be forecast by the method of hydroelectrothermal analogs, or by approximate formulas, in accordance with the recommendations of Appendix 2.

5.10. Figure 2 shows recommended designs for frozen dams, with remarks.

The scheme of a dam with an air-permeable cooling layer [Fig. 2 (f)]* has not yet been realized. The design makes it possible to raise the head on the dam gradually and is applicable for heads up to 10 m.

*Requirements on designs for frozen dams*

5.11. The central parts of frozen dams should be constructed of densely-placed, thawed, cohesive soil (clay loam or sandy loam) that can retain in its pores the water needed to form an impervious ice-soil core.

5.12. To strengthen the heat-insulation effect of the upstream shell, the frozen dam core should be placed at a sufficient distance from the upper pool. To increase the volume (thickness) of the upstream shell,

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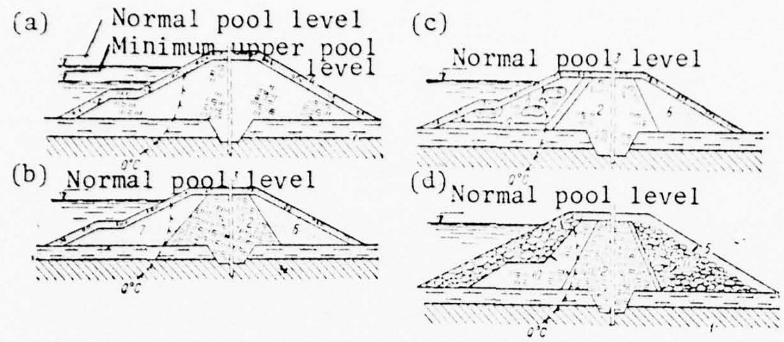
*USSR Inventor's Certificate 383,775; *Byull. izobret.*, 1973 (24).



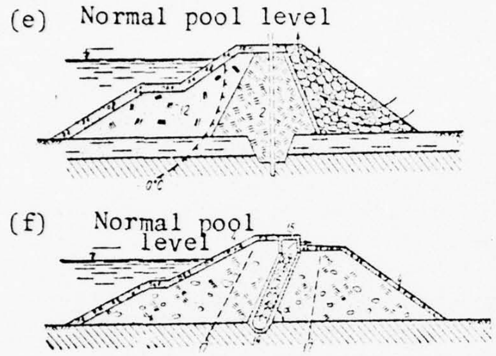




Fig. 2. Recommended designs for frozen dams.
(a) Homogeneous dam from cohesive soil dumped in thawed state, without inclusions of peat and frozen clods. (d) Dam with frozen core from cohesive soil, with upstream and downstream shells from any thawed soils. (c) The same, with inclusions of frozen clods in the upstream embankment. (d) Earth-rock dam with frozen core from cohesive soil and upstream heat-shielding shell. (e) Earth-rock dam with frozen core from cohesive soil, upstream shell from any local soils, and rockfill downstream shell (arrows show natural winter air convection in rockfill). (f) Dam with air-permeable cooling layer (arrows show forced leakage of cold winter air in pores of the homogeneous, coarse-grained layer).
(*1*) Permafrost foundation (top layer: settling ice-saturated soil); (*2*) frozen core from cohesive soil; (*3*) freezing columns of frozen cutoff (tentative); (*4*) shielding layer from sand, sandy gravel or crushed stone/gruss; (*5*) smoothing layer, reducing natural convection in rockfill; (*6*) downstream shell; (*7*) upstream shell; (*8*) inclusions of frozen clods; (*9*) transition-drain zone; (*10*) upstream heat-shielding shell; (*11*) downstream rockfill shell; (*12*) upstream shell; (*13*) transition zone; (*14*) perforated pipe (gallery, channel) for injection of cold air; (*15*) reinforced-concrete air collector channel; (*16*) coarse-grained cooling layer; (*17*) freezing limit.

wide berms should be included or the upstream slope should be flattened. Expanding the upstream shell also assures the stability of the dam after the inevitable thawing of the thaw-settling foundation under the upstream shell.

5.13. Natural taliks under the middle part of the cross-section of a frozen dam and under the downstream shell should be frozen ahead of time, before the reservoir is filled. The talik under the upstream shell, because it will inevitably thaw during the operating period, should be retained.

5.14. Up to 15% frozen soil can be placed in a homogeneous dam or in the frozen core of a zoned dam. Up to 30% frozen clods and up to 10% peat can be placed in the upstream shell. If there is a possibility that the downstream shell of a frozen dam will thaw, the inclusions of peat and frozen clods are limited just as in the upstream shell. If the internal zones are safe from thawing during both erection and operation of a frozen dam, the content of frozen clods in the downstream shell is not limited, but peat (even frozen) is limited to 20%. Inclusions of peat and frozen clods should be distributed uniformly through the volume of soil enclosing them, not concentrated in separate places or in layers.

5.15. In a frozen dam, soils to be placed in the upstream shell, which will be thawed under operating conditions, should have sufficient shear strength after thawing and consolidation; the deformability of these soils should lie within such a range that the thawed part of the dam cross-section will remain monolithic when the foundation soils thaw and settle.

5.16. In order to assure the imperviousness of frozen zones and the thermal stability of the dam, the mean temperature of the frozen core and of the cooled permafrost soils of the foundation and abutments when the reservoir is filled should lie 1.5°C below the phase-transition temperature of moisture in these soils.

5.17. The central frozen zone of a frozen dam should be completely monolithic and impervious. The presence of cracks and voids, or of zones of free-flowing frozen soils with pores incompletely filled by textured ice, is not permitted. To satisfy these requirements, a soil should have a moisture content near 0.9 times its total moisture capacity $W_t$ at the planned density of placement $\gamma_{opt}$.

5.18. The frozen core should be and continue to be a continuous, impervious barrier making contact with the permafrost foundation. The productivity of the cooling systems and the spacing between freezing columns used to build up the necessary cold reserve in the dam should be calculated in accordance with Appendix 2.

5.19. At the moment when the reservoir is filled and in the first years of service, the frozen zone may be somewhat smaller than the limiting values, provided there is a system for forced cooling of the dam to

assure complete imperviousness. The frozen-core thickness sufficient to take up the head can be approximately determined in accordance with Appendix 5.

5.20. The contact of a frozen dam with the valley sides should be made in such a way that the frozen zone of the embankment touches the permafrost soils of the sides. To this end, the frozen cutoff should be continued into the valley side to a distance determined by thermotechnical solution of the problem of soil thawing on the contact section (Appendix 2).

5.21. Thawing of ground ice should be considered one of the main reasons for the destructive concentration of seepage on thermokarst cavities and for the dangerous deformations of frozen dams. In the planning and construction of frozen dams on loose sediments enclosing ice lenses and wedges, it is necessary to allow for the thawing of large ice inclusions in the foundation and the abutments of the thawed upstream shell of the dam, and also within the region to which the thermal effect of the reservoir extends.

If impervious elements of a frozen dam (cores, membranes, frozen cutoffs) must make contact with high-ice-content soils in the bank slopes, these elements should be cut down to water-resistant permafrosts with low settlements. The impervious element should be continued in the bank beyond the predicted limit of reworking of the ice-saturated banks, or else the design should protect these banks from reworking (Appendix 6).

5.22. Ice-saturated banks with large ice inclusions where they abut structures of the pressure front can be protected against thawing by a layer of impervious soil (or pervious soil with a film lining) covered with filter material and rock or rubble from the wave action of the reservoir.

5.23. To reduce settlements and deformations of the upstream shell of a frozen dam, high-ice-content sediments near the upstream toe should be pre-thawed or replaced with thawed, compacted soils in the design thawing zone. Replacement will be most effective when the roof of the low-settlement permafrosts lies comparatively shallow (up to 10 m).

5.24. The downstream shell of a frozen dam may be erected on icy foundation permafrosts without special treatment of these soils (thawing before construction, extra freezing, etc.). The layer of vegetation should be removed.

5.25. Water-filled depressions are not permitted in the lower pool near the downstream toe of a frozen dam. Water appearing there (from bank seepage or melting of snow and ice) must be removed from the toe of the dam.

5.26. Plans for a frozen dam should take into account that, in regions where ice wedges and other forms of ground ice exist, the permafrost may thaw through under the reservoir bottom.

5.27. The crest, downstream slope and freeboard of a frozen dam are subject to freeze-thaw cycles. To prevent heaving, frost splitting and solifluction slides on the downstream slope, the seasonal freeze-thaw layer should be made from dumped sands, gravels and crushed stone, with no silt or clay admixture (Appendix 7).

5.28. The steepness of the upstream slope of a frozen dam should be calculated for complete settlement of the foundation under the upstream shell, that is, with a reserve assuring stability of the upstream slope while and after the thermal action of the reservoir thaws the foundation under the upstream shell of the dam.

5.29. Forced cooling of the dam by a system of air-cooled freezing columns should be provided for the stabilization period of the design temperature regime in the frozen part of the dam. Afterward, it should be possible to cut off the forced cooling and preserve the freezing system so that it will be operable in emergencies.

5.30. If the downstream shell is constructed from porous, coarse-grained material (rockfill, shingle or rubble), the core of the frozen dam should be dumped from clay loam containing not less than 40% silt and clay particles.

Thawed dams

5.31. Thawed dams in the Far North must meet all requirements set down in existing standards documents for dams from earth materials in the temperate zone (Paragraph 1.2).

5.32. The following design schemes can be used in the planning of thawed dams:

(a) Homogeneous filter dam from cohesive or cohesionless soil. Must contain internal heated drainage and a protective layer to prevent frost heaving, splitting, and solifluction and landslides in the seasonal freeze-thaw layer on the crest, downstream slope and freeboard. This type of dam is recommended for non-settling foundations of types (a) and (b) (Paragraph 4.6) that remain sufficiently water-resistant after thawing.

(b) Dam with a central clay core (Fig. 4), with a lining or upstream shell from clayey soil [Fig. 3 (b)] and the shells from cohesionless soils or rockfill. Such dams may be erected on non-settling foundations or on settling foundations of limited strength, provided the thaw-settling layer under the base of the dam is thawed before construction.

(c) Dam with a rigid central membrane tightly sealed to the foundation. The foundation does not deform; it retains its water-confining properties after thawing. A necessary design element in this type of dam is the sand core, which protects the membrane from damage when the shells settle [Fig. 3 (a)].

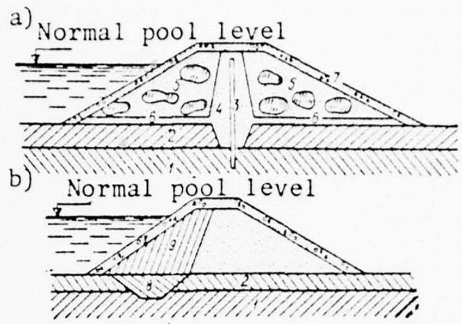



Fig. 3. Thawed dams, (a) with rigid membrane and (b) with upstream shell or lining. (*1*) Frozen water-confining stratum with no thaw settlement; (*2*) layer of high-ice-content soil with thaw settlement; (*3*) rigid membrane; (*4*) sand core; (*5*) shells from any local soils; (*6*) drain layer from local pervious soil; (*7*) shielding layer from non-heaving soil; (*8*) clay cutoff; (*9*) upstream shell or lining from clay.

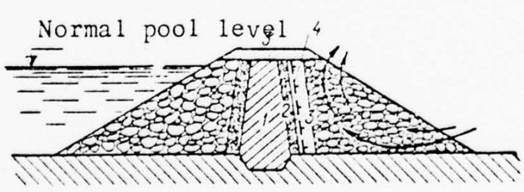



Fig. 4. Heat-shielding layer in a thawed rockfill core dam. (*1*) Thawed core; (*2*) drainage system; (*3*) heat-shielding layer; (*4*) limit of maximum freezing of heat-shielding layer. Arrows show natural winter convection in the downstream shell.

(d) Earth-rock dam with a core or membrane from cohesive soil. The dam must contain heat-insulation layers from any non-heaving local soil (except peat) that will (1) protect the top of the core (membrane) from deep seasonal freezing, heaving, and frost splitting and (2) provide normal operation of the internal drainage when natural convection of cold air cools the downstream rockfill shell during the winter (Fig. 4).

(e) Dam with flexible impervious elements from polymer materials, protected against drastic temperature variations by the heat-shielding load of a crest from non-heaving soils (Fig. 5 and Appendix 7).

*Note. The designs shown in Fig. 5 for dams with flexible corrugated membranes, internal linings and blankets were worked out on the basis of trials of film linings in temporary, low-head dams (heights up to 8 m) in Far North regions, and also the results of special model studies conducted at the Krasnoyarsk Promstroyniiproyekt in 1965-67.*

5.33. The use of flexible impervious elements made of polymer films is recommended in dams on foundations of types (a), (b), (c) and (d) (Paragraph 4.6), provided the soils are thawed, or the surface layer of ice-saturated soil is removed, in the foundation of the membrane and blanket. This layer in the foundation of the upstream and downstream shells may be allowed to thaw naturally when the dam is in service. In this case, dumped-sand zones around the flexible vertical impervious elements should be used to take up the settlements of the foundation as it thaws and to protect the membrane from deformations of the upstream and downstream shells.

5.34. The most suitable polymer material for the construction of permanent impervious elements in the north is polyethylene film stabilized with 2% carbon black, 0.2-0.4 mm thick, in rolls. This film resists freezing down to -70°C (203 K) and has an ultimate elongation of 600-800% and a tensile strength of about 120 kgf/cm$^2$ (12 MPa). It retains its properties without change for many years in an aqueous or soil medium, or with slight changes over several days in the open air and under the action of solar radiation.

The best method for joining polyethylene sheets is welding. The strength of a welded joint amounts to 50-100% of the film strength. If welding equipment is unavailable, or if adverse conditions prevent high-quality welding, then the sheets may be joined by taping (with adhesive polymer tape) after the edges have been folded together three to five times. The edges of sheets can also be joined with freeze-resistant mastic.

Temporary impervious elements, intended for one to three years' service, may be made of other polymer materials. The selection may depend on technical and economic considerations and local conditions.

5.35. It is most desirable to use dams with flexible polymeric impervious elements when no high-quality clays exist near the site, or

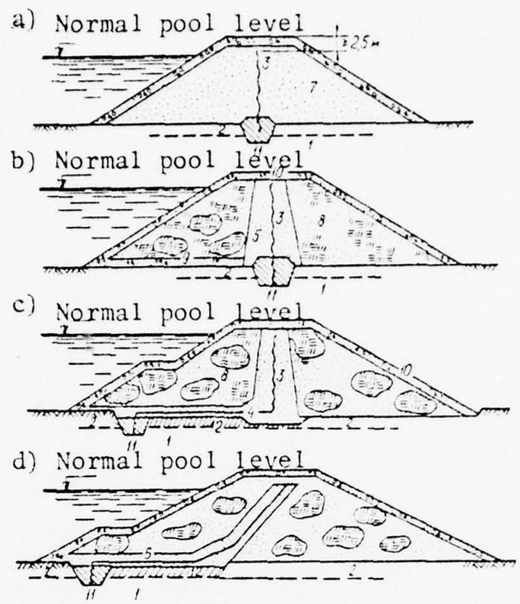



Fig. 5. Recommended designs for thawed non-filter dams with flexible impervious elements made of polymer films.
(a) Homogeneous dam with central membrane; (b) dam with central membrane inside a protective sand core; (c) dam with central membrane and internal blanket; (d) dam with internal inclined lining and blanket.
(*1*) Permafrost water-confining layer with no thaw settlement; (*2*) ice-saturated thaw-settling foundation layer; (*3*) flexible corrugated membrane made of polymer film; (*4*) flexible blanket; (*5*) protective core from sand or sand-gravel mixture with rounded grains; (*6*) protective layer from sand or sandy gravel; (*7*) dam embankment from sand (sandy gravel); (*8*) upstream and downstream shells; (*9*) drain layer from coarse-grained soils; (*10*) protective layer from cohesionless soil with varying grain size; (*11*) clay-loam cutoff; (*12*) zone of preconstruction thawing of a layer of high-ice-content soil.

when such soils are available but converting them from the ice-saturated to the waterlogged thawed state would greatly complicate the process of preparation, dumping and compaction (especially in a rainy summer).

5.36. To protect the film-type impervious element from harm, sand or sand-gravel mixture should be placed in a layer not less than 0.3 m thick under the film and a protective layer not less than 0.5 m thick. The underlayer may consist of rounded particles not larger than 6 mm, the protective layer of particles not larger than 40 mm. Reliable performance of the film membrane is aided by free placement of the film (without tensioning) on horizontal or slightly sloping underlayers, and by corrugation in vertical or steeply sloping impervious elements [Fig. 5 (a, b, c)]. No frozen clods or accumulations of coarse particles are permitted at the downstream contact with the film.

5.37. Impervious elements should not extend into the region where temperatures alternate between above and below freezing. Seasonal freeze-thaw cycling is allowable only in the heat-insulation layer on the crest and downstream slope. This layer prevents the appearance of dangerous cryogenic processes--heaving, frost splitting, and solifluction and landslides--in the impervious and other elements of the dam embankment.

5.38. Practically any kind of soil can be placed in the shells. However, the quantity of incompletely decayed plant remains should not be greater than 10% of the total soil volume, and the content of frozen clods should not be greater than 30%. Up to 15% frozen clods can be included, even in impervious elements from soils. It is just necessary to make sure that the frozen inclusions are not concentrated in one place but are distributed uniformly through the volume.

5.39. Clay impervious elements and the foundation may be connected by a clay cutoff comprising the lower part of the core, lining or upstream shell, or by a grout curtain. A rigid membrane, a flexible central membrane or an internal polymer-film blanket should be connected to the foundation with a densely-compacted clay-loam or clay cutoff (Figs. 3, 4, 5). The clay cutoff (curtain key) should extend not less than 1 m into the solid rock or non-settling water-confining layer.

5.40. The following recommendations are made for reducing the deformations of a thawed dam:

Preliminary thawing and compaction of the thaw-settling ice-saturated foundation layer during the construction of the dam.

Combined thawing, including (a) rapid thawing before construction and (b) natural completion of thawing by the thermal action of water and by seepage, after the filling of the reservoir and during the first years of operation of the dam.

5.41. When a dam is planned on a thick layer of loose soil or on jointed rock whose permeability rises sharply when it thaws, a grout curtain should be included. Grouting can take place after artificial

preliminary thawing of the cutoff foundation reaches the calculated depth to which the foundation will thaw. The foundation may be grouted step-by-step (layer-by-layer) as it naturally thaws, over the first few years of service of the reservoir. This procedure requires constant observation of the foundation-rock temperature and thawing.

5.42. The cutoff and the grout curtain should intersect all the foundation materials, including settling soils, rock and rocky soils with joints filled with a pervious material or with ice, and non-rocky soils (including ice veins and incompletely decayed organic remains).

5.43. For a core from clay loam with a crushed-stone phase, containing a significant quantity of silty particles, the thickness should be not less than $0.25H$, where $H$ is the maximum design head of water on the dam. For a more homogeneous clay loam, containing up to 30% large particles ($d \geq 2$ mm), the minimum core thickness can be taken equal to $0.1H$.

5.44. Frozen jointed rock can be used without special foundation treatment to confine water only when the joints are filled with clay or the joint volume is so small that, when the ice in the joints melts, the permeability of the rock rises only a little and there is no danger that the impervious-soil element will erode where it makes contact with the joint. If this requirement is not met, it is necessary to provide for grouting, or another type of injection sealing, of the thawing zone of the foundation.

5.45. If the joint system in the foundation rocks connects directly with the upper pool, then leakage can be reduced by the following method: After the reservoir is filled, clay loam (possibly including frozen soil) is dumped into the water to form a blanket and partly silt over the zone where joints appear at the surface. The outlets of joints to the surface within the downstream shell should be protected by a layer of filter material to prevent a leakage current in the foundation from carrying off the silting material when the foundation thaws.

5.46. The top of the lining (core) of the dam should be protected against frost splitting by a layer of sandy gravel or soil with crushed stone. The minimum thickness of this layer should equal the depth of seasonal frost penetration. In the planning of dams higher than 10 m, the necessary thickness and design of the protective layer should be refined on experimental plots.

5.47. The length, cut depth and design of a bank contacting impervious element are determined from the predicted thawing of the bank slope (thermotechnical calculation), the predicted development of seepage around the dam, and the predicted development of thermokarst and bank thermal abrasion on icy portions of the valley walls near where they abut the dam and other pressure structures of the water-engineering system.

If there is appropriate technical and economic justification, ice-saturated soils in the abutments that would be unstable on thawing may

be either thawed and compacted or replaced before construction.

5.48. If a thawed dam is planned on a foundation with a large thaw settlement, the soil in the upper layer (the most highly ice-saturated layer) should be either thawed and compacted or replaced before construction starts. If a high-ice-content foundation thaws naturally, the design of the dam cross-section should make provision for accommodating slow deformations of the foundation as it thaws. This requirement is met by shells from free-flowing soils with varying grain size, with shallow slopes.

5.49. If a thawed dam is planned for a frozen foundation with no appreciable thaw settlement, natural thawing of permafrosts should be taken as a guide. This process usually goes to completion in the initial service period of the dam (not more than 10-12 years).

5.50. A settling foundation of limited thickness may be pre-thawed by temporary filling of the reservoir before the dam is completed but after the spillway works are completely finished. After the foundation stops thaw-settling, the reservoir is emptied, the deformed parts of the embankment are fixed in place, and placement is continued until the embankment cross-section is complete.

5.51. In the initial service period of a thawed dam on a foundation of type (c) or (d) (Paragraph 4.6), the foundation soils will settle as they thaw. Provision should be made for immediate fixing of the two principal crack systems that may then develop in the embankment. Special attention should be paid to the longitudinal cracks in the upstream slope along the water line, and to the cracks transverse to the axis of the structure, in the parts of the thawed-dam embankment that abut the frozen soils in the valley sides.

5.52. The freeboard (above normal backwater level) should be calculated to allow for thaw settlement of the foundation, unless the plans include pre-thawing or replacement of the ice-saturated layer of the foundation with compacted clay.

5.53. The permeability properties of soils in most regions of the north have not received enough study; in addition, these properties vary with thawing and freezing. Thus, generally-accepted methods for checking the permeability and selecting the filter materials should be used only for dams not more than 6-8 m high. For more crucial structures, special permeability modeling is needed, using soil samples with the primary cryogenic texture, which changes in the process of thawing.

5.54. Thaw-settling frozen soils that are slightly pervious after thawing may be left in the foundation of a thawed dam. Vertical sand drains can be used to increase the bearing capacity and speed up the consolidation of the soil as it thaws. Water from the sand drains should be led to a heated drainage system inside the downstream shell.

5.55. To prevent the formation of icings, water is not allowed to

escape freely on the downstream slope, on the bank slopes where the dam abuts the valley walls, or into the body of an unheated rockfill downstream shell.

5.56. The organized discharge of drainage water into the lower pool should be protected from freezing. If there is no water in the lower pool, or if it has frozen through in the winter, a heated catchment pool should be constructed. From the pool, the drainage waters should pass into a water inlet or should be automatically pumped into the upper pool.

5.57. Internal drainage in an earth dam should be situated in the permanently thawed part of the cross-section. It should be far enough from the downstream slope that that part of the downstream shell that freezes through will retain its stability when the total hydrostatic pressure is transmitted to it from the upper pool if the drainage system accidentally freezes. This requirement is best satisfied by positioning of the drainage where the downstream face of the impervious element makes contact with the foundation.

5.58. In dams 15-25 m high where the downstream shell has a large content (20% or more by weight) of fines ($d < 2$ mm), the lower third of the downstream slope should have a supplementary drainage system.* The purpose of the system is to protect the downstream slope, in case the internal drainage system accidentally freezes, from erosion by drainage water percolating around the internal drains. The thickness of the coarse-grained portion of the filter ($d \geq 2$ mm) should exceed the depth of seasonal freezing (on the order of 3-3.5 m). If it is not economical to install such a thick filter, or if the lack of local materials makes it impossible to do so, the thickness of the filter layers is chosen without regard to seasonal freezing. In this case, a heat-shielding layer, made of any local soils not subject to frost heaving, is required to prevent freezing of the drainage system.

5.59. In order to slow down freezing and reduce the seasonal temperature fluctuations due to convection within a downstream shell of coarse shingle or rockfill, a layer of pit-run fines or sandy gravel should be spread over the downstream slope, and an interlayer of fines should be left in the shell embankment. These steps prevent air convection in the rockfill voids, exchange of air in the voids with the atmosphere, and the formation of internal icings.

5.60. The heat-shielding layer, which prevents freezing through in (a) the internal drainage system of a dam with a rockfill downstream shell or (b) the external (emergency) supplementary drainage system of an earth dam, should have such a thickness that water entering the drainage system cannot freeze when the dam slope and the downstream shell are

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*Translator's note: Original has *naslonnyy drenazh*, where the first word does not appear in any source available to me. I think it is a misprint for *nasloyennyy* 'added on.'

cooled in the winter. The heat-shielding layer should maintain a permanently thawed zone not less than 0.5 m thick adjoining the drainage system, to prevent seepage water from coming into contact with the freezing zone of the heat-shielding layer.

5.61. The selection of materials for wave protection of the dam and abutments and the selection of filters to prevent particles and aggregates of cohesive soil from being carried into the fill should be made in accordance with the Chapter "Dams from earth materials" in the Construction Standards and Regulations.

6. Requirements on spillway works

6.1. Spillway and inlet works in a water-engineering system that includes a frozen dam should, as a rule, lie outside the dam itself (see Paragraph 4.16).

6.2. Especially unfavorable for the construction of bypass spillways of the self-regulating canal type are bank terraces made up of high-ice-content settling soils that include ice wedges or layers. If these are present, the construction of a bypass spillway canal is not recommended. A siphon spillway can be used for small flows; for sizable flood flows, a chute-type spillway should be constructed. The latter type is supported by piles from the frozen dam embankment.

6.3. A spillway cutting through the embankment of a frozen dam or supported on the surface of the frozen portion of the dam should be thermally insulated. The insulation should minimize the local heating action on the dam when the spillway is discharging water. The zero-degree (Celsius) isotherm should not go outside the material of the spillway or thermal insulation. Local leaks through the spillway embankment are not permissible. The thickness of the heat-shielding layer is determined by a thermotechnical calculation. Roughly speaking, the thickness of the layer of fine-grained soil is 2-2.5 m for a spillway in continuous operation.

6.4. If the path of a spillway channel passes through heaving soils, heat shielding from non-heaving soil, 1.0-2.5 m thick, should be placed under the bottom and sides of the canal.

6.5. If the path of a spillway for large flows can be laid out only through a high-ice-content permafrost foundation with large inclusions of pure ice, the walls and bottom should have heat insulation and waterproofing. Figure 6 shows one possible design for a spillway canal. Lining *1*, made of polyethylene film (Paragraphs 5.34 and 5.36), provides reliable waterproofing for the icy foundation. The thickness of drainage and protective layer *2* should be not less than 0.5 m. A layer of non-heaving soil *3* with clay filler protects the icy foundation from seasonal thawing. The thickness of this layer, determined by a thermotechnical calculation, should be not less than 1 m. The drainage layer must be dried after the water discharge stops.

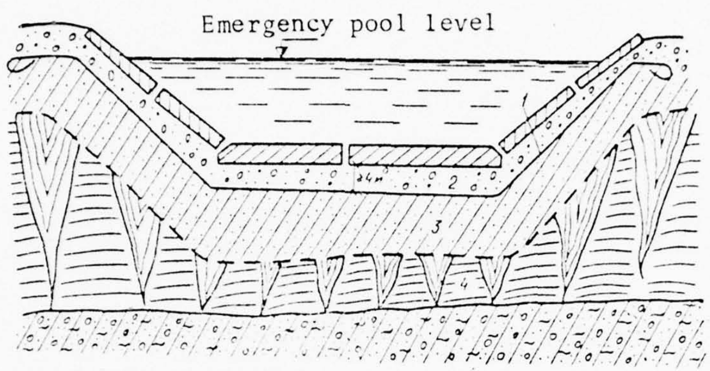


Fig. 6. Thaw protection of the icy foundation of a bypass spillway. (*1*) Film lining; (*2*) protective and drainage layer from gravel and shingle; (*3*) heat-shielding layer from non-heaving soil; (*4*) icy foundation soil.

6.6. The film lining or membrane of a dam should be carefully cemented to any spillway or dewatering conduit that intersects the dam, all the way up to the emergency pool elevation. The design of the joining of film and discharge works should make it possible for the structure and the soil to shift relative to each other.

6.7. Under the conditions of the Far North, radial gates are the most promising type. These gates have supports that never go under water, and they can easily be fitted with heaters.

7. Construction work

Preliminary work

7.1. Preliminary work and construction work have a vital goal of preserving natural conditions as far as possible. In regions of buried ice and ice-saturated soils with large thaw settlements, the surface layer of moss and vegetation should not be disturbed where structures of the pressure front abut the banks and at the waterline of the future reservoir.

7.2. Surveying and construction work should be carried on with the minimum damage to the layer of moss and vegetation; machinery should be moved primarily at levels to be flooded or on soils that do not settle.

7.3. In preparation of the reservoir bed for flooding, the layer of moss and vegetation and any incompletely decayed floating peat should be removed. The material removed should be placed in piles above the waterline of the future reservoir. Allowance should be made for changes in surface level resulting from thawing of buried ice and settlement of icy soils after the reservoir has been filled. The piles should be protected from washing away by surface waters.

7.4. On solifluction banks of the reservoir, piles of material taken from the surface of the reservoir bed should be placed on the slope transverse to the direction of solifluction flow of the ground. These piles should not hinder surface drainage, that is, should not form stagnant ponds of water. After they have naturally frozen through and linked with the permafrost stratum in the bank slope, the frozen piles will take up the pressure when solifluction sliding takes place in the soil layer.

## Foundation treatment

7.5. Foundations under thawed dams must meet all requirements applying to foundations in temperate climates, but are distinguished by a number of special features.

Specific types of work needed for foundation treatment for each type of dam were presented in Section 5. The present section examines possible ways of carrying out this work.

7.6. Grouting of frozen rock in the foundation of an impervious element (under the cutoff of core or lining) where the joints are filled with a pervious material and ice should be carried out after preliminary thawing.

7.7. The frozen rock of the foundation can be thawed either by the hydraulic method (injection of water into a hole, with the water running out through the space around the pipe at first and later flowing away through the thawed joints), or by steam heat. In the second case, a considerable amount of heat is liberated in the hole by the condensation of steam.

Ice-saturated, non-rocky soils containing significant amounts of clayey and silty particles (20% or more of particles with $d < 0.05$ mm) can be thawed only by methods that maintain or reduce the natural moisture content. Examples are the action of solar radiation under a transparent polymer film; ac electric heating; dc electric heating using the electroosmotic effect to partly dewater the waterlogged soils; exhaust gases from jet engines; and steam heating with sealed steam pipes.

7.8. The construction area can be temporarily drained and seasonal melt waters can be carried off when the ice-saturated soils are exposed by drainage ditches; but this can be done only through the construction of careful, two-layer heat shielding and waterproofing for these ditches. The linings should be made of clays, moss, peat or slag, or polymer films should be used.

7.9. Foundation treatment under a frozen dam depends to a great extent on the method used to erect the dam. If the dam is intended to freeze through after being built by placement of thawed soils, then the foundation treatment should be done just as under a thawed dam.

Under the downstream shell of a dam frozen layer-by-layer, it is sufficient to remove just the layer of moss and vegetation. Excavation under the cutoff of such a dam should cut through all the highly pervious foundation rocks (gravels and shingles, rock with joints of significant width) and large ice inclusions (veins and lenses). The layers of the cutoff and embankment of the dam should be frozen to the thickness of the seasonal-thawing layer until the onset of above-freezing mean daily temperatures, in order to prevent thawing of the icy foundation.

## Working of borrow pits and preparation of soil for placement

### *Cohesive soils*

7.10. A cohesive soil in a permafrost zone is worked by removal of the top thawed layer with scrapers or bulldozers, or removal by graders followed by collection of the soil into winter storage piles. Scrapers can work soils with moisture contents not above optimum (so as to avoid adherence to the bucket); the soils are then placed in the embankment without intermediate storage.

7.11. To speed up the thawing of a cohesive permafrost soil, it is useful to treat the top layer with a tractor-type ripper.

7.12. Electroosmosis and the electric heating that goes along with it can also be used to speed up the thawing of soil in a borrow pit. To start the process, the frozen ground from the surface between electrodes of opposite sign should be salted, since frozen ground is an insulator. The salt dose is arrived at by experiment. For soil to be placed in a thawed dam, the salt dose should yield the maximum rate of thawing and drying. For soil to be placed in a frozen dam, the dose should be a minimum, just enough to start the process; afterward, the process goes because of the natural salt content in the thawing soil. The outdoor-air temperature at which this method effectively heats and dries a

cohesive soil in the borrow pit is also determined by experiment (see Section 3).

7.13. Other methods suitable for thawing cohesive soil in a borrow pit include: steam thawing, with a sealed system of steam wells; thawing by heat of exhaust gases from jet engines; ac electric thawing; thawing by solar radiation under film coverings; and use of hot air from a heater with a fuel nozzle. None of these methods increases the moisture content of the soil.

7.14. Salting the soil also speeds up the thawing of frozen cohesive soils and lowers the temperature at which they freeze. However, a cohesive soil can be salted only if it will go into a thawed dam.

7.15. All the methods for speeding up the thawing of frozen cohesive soils not only increase the working speed but also extend the working season. The choice of a method for thawing soil depends on the technical and economic calculation and the local conditions; the efficiency and operating parameters are determined in a trial thawing.

7.16. A waterlogged cohesive soil should be dried before placement in the dam. Drying can take place either in winter storage piles at the site where thawed soils are prepared for winter storage, or at the edge of a plot on the dam, provided the zones adjacent to the plot do not allow water to emerge onto the surface of cohesive soil placed in the dam earlier.

7.17. To lengthen the working season and reduce the effects of atmospheric factors on the temperature and moisture content of the worked soil, inflatable or framework tents or film coverings can be used.

7.18. Electroosmosis can be used to speed up the drying of soil in winter storage piles. The placement of electrodes, the voltage and current, and the method used to drain off the water should be chosen by experiment.

7.19. To avoid waterlogging by surface waters, winter storage piles of cohesive soil are placed along the local slope. It is useful to place a thin (0.3-0.5 m) layer of pervious soil at the foundation to remove excess moisture from the piled soil.

7.20. Winter storage piles of cohesive soil should be not less than 10 m tall, so that significant volumes of the piles will not freeze through. In placing the piles, methods that would compact the cohesive soil should be avoided. A layer of "peno-led" [see the note below] can be placed over the winter storage pile at the onset of below-freezing temperatures, in order to reduce the freezing depth. The thickness of the layer should be the calculated required value. In the spring, before temperatures go above freezing, the layer should be removed from the surface of the pile. The same end can be achieved with quick-hardening foam, which can be removed from the storage pile immediately before the pile is worked.

*Note. "Peno-led" is a heat-insulation material made from air, water, and 2-3% of a foaming agent. It is produced in fire engines or in special equipment. Peno-led is applied in layers to the surface to be shielded, at an air temperature of -10°C to -15°C (258-263 K). It cannot be walked on.*

7.21. To reduce the cooling surface, cohesive soil may be placed for the winter in an exhausted, dry face of the rock borrow pit. The drainage of waterlogged cohesive soil can be improved by the placing of a thin layer of draining soil on the bottom of the face; drainage of water from this layer is mandatory.

7.22. In the preparation of cohesive soil in winter storage piles for winter placement in the impervious element of a dam, it is necessary to calculate the minimum soil temperature for placement in the pile. This result must also be checked on an experimental layer. The aim of the procedure is to provide the soil with the heat reserve it needs to remain thawed while the storage pile is worked and while the soil is transported, distributed over the dumped layer, and compacted.

7.23. So that equipment in the borrow pit will not sink into the liquefied ground when it thaws, coarse-grained soil (rubble or shingle) should be prepared. This soil should be able to make a rigid skeleton on roads and parking areas used by machinery. For this reason, it is desirable to place excavators and transport in the borrow pit immediately on the bottom of the soil layer being worked.

7.24. Waterlogged cohesive soil that has liquefied on thawing can be made placeable by the addition of dry, loose soil. This can be done either right in the borrow pit or in the intermediate-storage pile. The only requirement is that the mixture have sufficient water-resistance after this operation.

*Pervious soils*

7.25. Dry soils with gravel-shingle and crushed-stone/gruss fractions, which remain free-flowing at temperatures below freezing, are worked and placed in dams the year round.

7.26. Soils of these types from water-saturated taliks under stream beds are always ready for working, but require some drying before they can be placed in the dam. The outdoor-air temperature at which soil taken from under water does not freeze depends on the properties of the soil and on local conditions. This temperature is established by experiment. Excavation of pervious soil under water is halted when the outdoor temperature goes below this limit.

7.27. The working of icy soil that would become pervious on thawing requires preliminary thawing. The following methods are most effective: hydrothawing, steam thawing, and thawing by solar radiation under a transparent polymer film in conjunction with sprinkling. After excavation, this type of soil requires drying. So that work can go on without

interruption, pervious soils are prepared for winter in storage piles on areas that will not be flooded.

*Rocky and semi-rocky soils*

7.28. Rocky and semi-rocky soils should be worked in accordance with all the requirements on this work in temperate climates. Permafrost affects the selection of drilling conditions and of the method for removing the broken rock particles from the bottom of the hole. Finely broken rock may decompose upon thawing.

7.29. If the work goes better in a dry hole, with the surrounding rock remaining frozen, then the bit should turn slowly and the bottom should be cleaned by blowing.

7.30. If it will be more productive to drill in thawed rock, the bit rotation speed is not limited, and drilling fluid is supplied to the bottom at a temperature above freezing and with a heat reserve against its own freezing. If necessary, the drilling fluid can be heated in an electric boiler (a heater surrounding part of the pipe through which the fluid is supplied to the hole).

7.31. In every case, the choice of method and equipment for working rock depends on technical and economic calculations and local conditions. The efficiency of the method used and the mining process are verified by experiment. Dimensions of pits and the siting of roads should be in accordance with the Chapter on "Earth structures" in the "Construction Standards and Regulations."

Placement of soil in the dam

7.32. In the construction, in the Far North, of thawed dams and dams constructed as thawed but intended for freezing after completion, all the requirements on dams in temperate climates must be fulfilled. However, the severe climatic conditions and the presence of permafrost in the foundation impose a number of additional requirements.

7.33. To prevent the collapse of frozen, non-rocky walls of the cutoff trench of the dam, it is best to fill the trench with soil before the walls start to thaw. To this end, the trench should be cut in the frozen soil at the onset of below-freezing mean daily temperatures in the autumn, or else at the end of winter, after the last freeze that would prevent the placement of thawed soil. The cutoff trencn should be filled with soil where it passes through frozen, non-rocky soil and ice-heaved rock; this should take place before the onset of temperatures above freezing.

7.34. The air temperature below freezing that will permit the placement of prepared soil in the impervious element of the dam is determined by experiment. For instance, for a thawed dam where the travel distance is up to 3 km and the salt dose is 20 kg NaCl per cubic meter of soil, protection of the core surface in the period of extreme cold should begin

at temperatures below -20°C (253 K).

7.35. The residence time of loose soil on the dumped layer at air temperatures below freezing should also be found by experiment. Under the same conditions as in the preceding example, the residence time of soil before compaction is finished, at a temperature of -20°C (253 K), should not exceed 15-20 min.

7.36. The season when cohesive soil can be placed can be lengthened by the following methods:

Salting of the soil to lower its freezing point; allowed only for thawed dams.

Heating of truck beds with covers that are removed just before dumping.

Heating of truck beds with exhaust gas.

Erection of tents over the soil-storage pile and over the layer of soil dumped in the dam.

Placement of soil in a pond of heated water.

7.37. It is categorically forbidden to leave interlayers of ice, snow or frozen clods between the soil layers in impervious elements of thawed or frozen dams. Snow can be removed by blowing, with a compressor and hose. Ice, if it cannot be removed by mechanical means, can be melted off with a fuel-burner nozzle or a stream of exhaust gases from a jet engine.

7.38. To avoid the appearance of a continuous layer of incompletely compacted soil, the edges running transverse to the dam should be rolled with compacting equipment to a slope not steeper than 1 : 10.

7.39. On thawed sections of the foundation, thawed soil is placed in accordance with the usual requirements on the construction of dams in the temperate zone.

7.40. When thawed soil is placed in the winter, it is necessary to prevent dangerous ice formation due to migration of moisture as the waterlogged soil freezes through. Thus the moisture content of the soil should be not greater than $0.9W_p$, where $W_p$ is the plastic limit of the soil.

7.41 . When impervious elements and transition zones in thawed and frozen

dams are being erected, frozen clods may be placed in a scattered fashion and in a quantity not greater than 15%. The moisture content of soil in the clods should not exceed $W_p$.

7.42. If it is impossible to dry a trench, any soil (including cohesive soils) can be placed in water, either through an artificial window cut or melted through the ice or by the "pioneer" method (dumping of fill the whole width of the trench). There must be no ice lenses or layers under the water. These may be thawed out by passing the water from the trench through heaters. The minimum free height of the dumped layer above the water surface in the trench is 0.5 m; the optimum height is 0.8-1 m. A layer of morainal soil dumped into water may reach a thickness of 5-7 m, provided the density (determined from test corings after the placement) is high enough.

7.43. It is desirable to cover the pond surface with heat insulation in the winter, in order to significantly cut heat losses. Slabs of expanded polystyrene come well recommended. For the tightest seal of all, the pond surface may be covered with free-floating flat slabs in the shape of regular hexagons.

7.44. Coarse-grained and coarse rubbly soil may be protected from water flooding only when the temperature is above freezing, on a non-settling foundation, and only with the water allowed to flow away from the impervious element (toward the upper pool in the upstream shell, toward the lower pool in the downstream shell). So that the maximum density of cohesive soil can be attained in the impervious element, the transition zones should be dumped in advance.

*Management of the work*

7.45. It is impossible to provide small-scale hydraulic construction projects in every newly-developed region with enough specialists having the needed skills. For this reason, long-term plans should be worked out for hydrotechnical construction. These plans will be fulfilled by specialized trusts possessing the needed staff of specialists and stock of machinery. As the specialist work is completed on one water-engineering system, the specialists and equipment move on to the next, where all plans have been prepared and all the preceding phases of specialist work have been completed.

7.46. Specialized trusts for hydrotechnical construction should be provided with machinery for working in the Far North (dump trucks with heated beds, mobile soil-thawing and soil-freezing units, mobile power plants for electric thawing and electroosmotic dewatering of soils, units for fabricating tents and linings from polymer films, scraper winches for cleaning snow from slopes, etc.). All machinery should be fitted for air transport.

7.47. A specialized trust provided with a stock of mobile machinery can also carry out maintenance and emergency repair work on all water-engineering systems of the region.

8. Geotechnical inspection

8.1. The construction laboratory should be set up at the same time that working of borrow pits and foundation treatment begin. All types of geotechnical inspection necessary for dam construction in the temperate zone are also conducted in dam construction in the north.

8.2. The construction laboratory carries on the following types of work as required by specific problems in the construction of earth dams in the Far North:

Determination of specific heat, thermal conductivity and thermal diffusivity of all soils to be used in the dam, at the density and moisture content attained in placement, in the thawed, frozen and transition states, over various ranges of temperature.

Determination of strength and deformability properties of all soils to be used in the dam (angle of internal friction $\phi$, adhesion $C$, modulus of deformation $E$, and Poisson coefficient $\mu$), in the thawed, plastic-frozen and solid-frozen states, at the densities and moisture contents obtained by placement in the dam.

Determination of the ice content (by volume) of the frozen soil in the dam.

8.3. The following guidelines should be used for soil sampling during the construction of a thawed dam or a thawed element of a frozen dam:

For determination of density and moisture content of a soil in the impervious element (core or lining) or in the transition zone, one sample, 200-400 $m^3$ of soil as placed.

For determination of density, strength and deformation properties and grain-size distribution of rock (rock for fills and blankets, shingle) and other materials placed in the shells, one sample, 10,000 $m^3$.

For determination of grain-size distribution and permeability, strength and deformation properties of soils placed in the core and transition zones, one sample, 5-10,000 $m^3$.

The soil sampling time, air temperature and humidity, wind speed, precipitation rate, state of samples, and sampling point (height on the dam and position in plan) should be recorded in geotechnical test journals.

8.4. In the soils of the frozen zone of the dam, the contents of ice and unfrozen water must be determined, and it must be ascertained that there are no interlayers of pure ice.

8.5. In winter placement of fine sands and clays, the temperature in the middle of the layer at the end of compaction must be measured periodically. These measurements should be made when the quality of

borrow soil, the height of the face, the method of placement or the dimensions of the layer change, or when the air temperature or wind speed changes drastically. These measurements have the aim of assuring that all placement and compacting operations are conducted on thawed soil.

8.6. After borrow pits and trenches for the principal structures are opened, important departures from survey results should be recorded. If these departures might affect the design of the dam or other structures, the working drawings should be corrected accordingly.

8.7. If the borrow pits contain layers or benches of soil with greatly different properties from those specified, the mixing of soil in the borrow pit should be carefully checked until the mixture obtained is uniform and within specifications, and stratification of soil dumped in the embankment should be carefully watched for.

8.8. When construction on the dam is completed, tests should be performed to determine the density, moisture content, strength, deformability, permeability coefficients and grain-size distribution of all soils making up the dam and the foundation zone near the contact with the dam. It is desirable to do test borings at not fewer than three points across the dam axis and to take samples at least once per meter in each hole.

When a dam is constructed in several phases, a similar series of tests should be run for each section as it is finished.

8.9. The distribution of soil characteristics throughout the embankment, as determined by the laboratory during construction and by test borings in the completed embankment, should be placed with the "as-built" drawings before the dam is commissioned, and kept by the person in charge of the operating staff.

9. Calculations: Types and special features

9.1. In the planning of thawed dams to be constructed in the Far North, all calculations prescribed for dams in temperate climates should be performed. These are calculations of stability, seepage in the embankment and foundation, seepage around structures of the pressure front and settlements of the embankment and foundation during construction and operation, and hydrological and water-management calculations.

9.2. Calculations of the temperature regime of a thawed dam should include: calculation of hydrothawing of the base when there is leakage from the reservoir; calculation of the depth and rate of seasonal and long-term freezing of the downstream slope and crest of the dam, with allowance for leakage in the embankment and in the thawing foundation; calculations of the thicknesses of heat-shielding layers; calculations of the temperature regime of the impervious elements and drains, with and without allowance for the effect of the heat-shielding layers; and a forecast of changes in the cryogenic conditions resulting from the

thermal effect of the reservoir on the permafrosts in its bottom and banks.

*Note. Exact analytic and numerical methods for computer calculations of the temperature regimes of earth filter dams are quite complicated. Approximate, practical methods are in the first stages of development. The set of calculations set out in Paragraph 9.2 will be examined in detail in a future publication improving these Recommendations. See Appendix 4 for the results of a calculation of the temperature regimes of a thawed dam and a homogeneous frozen dam.*

9.3. On the basis of the calculations listed in Paragraph 9.2, a multiyear forecast of the temperature regime of the dam should be made up, and the time needed for temperature stabilization should be determined. The dynamics and the limiting (steady-state) position of the waterline in high-ice-content banks should be established for locations near the structures of the water-engineering system and other permanent structures (Appendix 6).

9.4. A single method, taking account of thaw settlements of icy foundations, should be used for stability calculations of the slopes of thawed dams, frozen dams constructed from thawed soils with subsequent freezing, and the upstream slopes of frozen dams constructed by layer-by-layer freezing. For a dam frozen layer-by-layer, the steepness of the downstream slope should be determined from the stability condition for a seasonally thawing heat-shielding layer.

9.5. Thermotechnical calculations of frozen dams should be carried out in accordance with Appendixes 2 and 3, using the calculated characteristics of soil, water, ice and air from Appendix 1.

9.6. In evaluating the results of calculations of the temperature regime and in analyzing the stability of a frozen dam, account should be taken of the annual drop in reservoir level in the course of the summer drawoff. This reduction in level results in freezing of the ground surface under the ice in shallow areas near the waterline. In these areas, the depth and rate of thawing will be less than the corresponding values computed for a constant normal pool level in the reservoir.

9.7. The calculation of the thermal regime of a frozen dam should carefully take into account the consequences of thawing of large ice inclusions in abutments not protected by a heat-shielding layer from the heating effect of the upper pool.

9.8. In preliminary, approximate calculations of the limiting thawing of the bottom and sides of the reservoir, it is permissible to assume that, if the contour of bottom and banks remains stable, the zero-degree (Celsius) isotherm of the steady-state temperature field (the boundary of the thaw basin) will not lie outside the vertical surface defined by the waterline.

9.9. For a dam with a frozen cutoff, stability calculations must be done for the initial period of the creation of the frozen core, which takes up the head until the downstream shell has finished freezing through (see Appendix 5).

9.10. Checking of the stability of the upstream and downstream slopes of a frozen dam must allow for the temperature dependence of soil-mechanical characteristics. The shape of the calculated sliding curves should take account of the fact that the probability of sliding of the upstream slope is greatest in the thawed zone of the embankment and foundation. A check is needed on the displacement of the thawed massif with respect to the frozen massif, with allowance for the sharp decrease in shear resistance of the ice-saturated soil of the foundation and bank slopes after thawing. The stability of the downstream slope must be calculated for the period of maximum seasonal thawing, at the highest calculated temperatures of the frozen ground making up the slope.

9.11. Calculating the stability, deformations and thermal regimes of dams at the planning stage requires data on the strength, deformation and thermophysical characteristics of frozen, freezing and thawing soils. For Class III and IV dams, and also in preliminary calculations for Class I and II dams, these data can be taken in accordance with Appendix 1.

9.12. In the evaluation of thaw settlement for an ice-saturated foundation containing ices at various levels and variously distributed in the thawing layer, it should be taken into account that the decisive factors in the thawing of ground ice are:

The decrease in volume (up to 10%) accompanying the phase transformation of ice to water.

The runoff of that volume of water that exceeds the total moisture capacity of the thawing rocks.

9.13. The settlement of an ice-saturated soil massif upon thawing of the reservoir bottom to the depth of penetration of the ice in veins bounding polygons can be approximately calculated by the formula

$$\Delta h_{tot} = \frac{\Delta V_k}{S} + \frac{\Delta V_{segr}}{S_p} K, \qquad (1)$$

where $\Delta V_k$ is the volumetric settlement of one polygon due to thawing of the ice in veins bounding polygons to depth $h$, m$^3$; $S$ is the mean area of one polygon bounded by the axes of ice veins, m$^2$; $\Delta V_{segr}$ is the volumetric settlement of the soil included between the veins of one polygon, due to thawing of segregated ice forming the texture of this soil, m$^3$; $S_p$ is the mean area of a polygon less the area of the ice veins forming it, m$^2$; and $K$ is a coefficient allowing for incomplete merging of voids after the thawing of ice inclusions ($0.2 < K < 0.9$), which depends on the lithology, size and number of ice interlayers.

*Note. If the dimensions in plan of the reservoir exceed 300 m, then all buried ice under its surface at depths greater than 1.5 m will thaw, and the settlement will be a maximum equal to $\Delta h_{tot}$.*

9.14. The mean relative thaw settlement of a high-ice-content soil can be calculated approximately (without allowance for consolidation) by the formula

$$\Delta h_{rel} = \frac{I_t + W_u - W_t}{100}, \tag{2}$$

where $I_t = I_{inc} + I_{cem}$ is the total ice content ($I_{inc}$ and $I_{cem}$ are the contents of ice as inclusions and ice as cement); $W_u$ is the moisture content as unfrozen water; and $W_t$ is the total moisture capacity of the soil after thawing.

All quantities appearing in Eq. (2) are determined as percentages of the total volume of the frozen massif under consideration.

9.15. After construction of the dam is complete, all calculations should be repeated in accordance with the as-built drawings, with allowance for the mechanical and thermophysical characteristics of the soils found by geotechnical inspection during the construction process. On the basis of these calculations, checks should be made of (a) the correspondence between design and actual characteristics of the materials and (b) the correspondence between deformations and temperatures in the dam as measured during construction and the design values for this period. The forecast of future behavior of the dam in operation should be refined.

10. Field observations

10.1. The entire series of field observations on water-engineering systems as specified for earth dams in the temperate zone should be made on dams in the Far North. These include observations of water levels in the upper and lower pools, ice thickness, flows, contents of mineral and organic matter in the water, seepage, settlements, transverse and longitudinal displacements, condition of dam slopes, and so on.

10.2. The piezometer design should allow head measurements at several points over the height of the dam. For measurements of the heads in partly frozen zones of the dam, antifreeze-filled piezometers in jackets of freeze-resistant resin should be used.

10.3. The footings of markers for settlement and displacement measurements on a dam from soil with a high fines content should all be placed below the seasonal freeze-thaw zone, and the markers themselves should be protected from freezing to the seasonally thawing soil.

10.4. Special features of the Far North make it necessary to carry out special observations:

Measurement of temperature in the embankment and foundation down

to a depth equal to twice the head on the dam.

Measurement of soil temperatures in the bottom and banks of the reservoir down to the calculated depth of thermal effect on the permafrost.

Measurement of water temperature over the reservoir depth (temperature-measuring wells are not carried up to the bottom of the ice, but are equipped with flexible hoses sealed at the ends, with floats; for winter observations, holes are maintained in the ice over the wells).

10.5. The sites of the temperature-measuring wells (at least three intersecting the dam, at least two in the abutments) should cover the whole dam and the adjacent part of the foundation for not less than 25 m above and below the axis of the dam and in the direction away from the abutments. The zone of observations should extend beyond the calculated zone of the thermal effect of the reservoir and structures.

10.6. The plans should provide for temperature observations to cover the foundation and abutments of all other structures on the pressure front of the water-engineering system (water intakes, spillways, dewatering conduits).

10.7. The upstream and downstream slopes and crest of the dam and all structures on the pressure front, as well as the foundation near the downstream toe of the dam, should receive regular visual inspection.

10.8. The point where a small quantity of water issues in the lower pool should be fitted with heated ducts and a low-voltage lighting unit for year-round observations.

10.9. The outlets from concentrated seepage must also be heated. The temperature, turbidity, flow, and chemical composition of the seepage water should be determined. The results of these observations should make it possible to determine whether the water has leaked from the reservoir or is interpermafrost water flowing from the banks through water-bearing strata. The water should be drained outside the water-engineering system, either into the upper pool or into a catchment pool, in order to avoid icings. If the water is unsuited for use, it is released beyond the limits of the system through a heated canal.

10.10. When the reservoir is first being filled and during floods and sharp variations in the temperature and seepage regimes of the dam and foundation, the whole series of field observations cited above should be performed daily. Graphs should be made up not less often than twice a week, showing the variations in measured values at the points where they change most rapidly.

When the water-engineering system is in stable operation, all these observations may be made once a week.

10.11. Heave meters should be installed at the crest of the impervious element of the dam and on all cohesive-soil elements of other structures in the system. The meters should reach to a depth not less than 1.5 times the seasonal freeze-thaw depth. They should be checked not less often than once a month. For a check on seasonal freezing and thawing, electric cryopedometers should be installed. These instruments use the resistance of the soil between electrodes to show whether the soil is frozen or thawed.

10.12. The following method should be used to ascertain that there are no cracks in the crest of the core (lining) in zones of longitudinal tension (in abutments and over projections and fractures in the foundation). For the first years of operation (until the dam has completely stopped settling), test trenches should be dug in the protective sand-gravel layer before each filling of the reservoir. These trenches should reach down to the crest of the core or lining.

Freedom from cracks in the abutments or between the walls of concrete or wood pressure-front structures and the soil of the dam can be determined by digging holes in the protective layer at the contact. If transverse cracks appear on the surface of the protective layer, the holes must be used to determine whether the cracks reach the level of the previous filling of the reservoir.

10.13. All inspection results and measurements are entered in a journal. The results of field observations must be analyzed before every arrival of a flood and before the winter filling of the reservoir. This analysis uses the piezometric head, temperature, deformation, and seepage-flow measurements and the list of defects noted to determine whether the dam is ready to take on head.

10.14. The results of the field observations are sent to the planning organization and to the supervising scientific research organization.

10.15. The methods and instruments used for field measurements and studies should be specified in the plan of disposition of monitoring and measuring apparatus and in instructions for field observations of the dam and other structures of the water-engineering system.

Appendix 1.
Strength, deformation and thermophysical characteristics of frozen, freezing and thawing soils

Field and laboratory methods for determining the strength, deformation and thermophysical properties of soils are complex and require large amounts of time and special, nonstandard equipment. For this reason, planning organizations cannot always use these methods.

It has been found that the physical-mechanical characteristics of thawed and frozen soils depend on the moisture content, bulk density and a number of other physical properties that are not very difficult to find. Thus, in parallel with methods for direct determination of the physical properties of soil (field and laboratory tests), indirect methods can also be used.

In accordance with the Chapter "Foundations and footings on permafrosts" in the Construction Standards and Regulations, the strength properties of frozen soils are determined as functions of the bulk density and total moisture content. This is in effect an indirect method.

Indirect procedures do not supplant but supplement the field and laboratory methods of study, and can be recommended for the gaining of starting data in structural design at the preliminary planning stage--the engineering-plan stage--or in the planning of temporary structures.

Starting values for the physical properties of frozen, freezing and thawing soils found by experiment make it possible to obtain other soil characteristics by calculation. Doing this requires a knowledge of the type of soil, its grain-size distribution, the characteristics of the ice inclusions (cryogenic texture), and the following quantities:

$W_t$, the total moisture content, a fraction.

$W_{inc}$, the moisture content as ice inclusions, a fraction.

$\gamma$, the specific gravity of the soil, g/cm$^3$.

$\gamma_{spec}$, the density of the soil, g/cm$^3$.

$t_0$, the natural temperature of the permafrost layer at a depth of 10 m (level of zero amplitude), °C.

Derived physical properties of soils, determined by calculation, include:

$\varepsilon_i$, the initial porosity coefficient, a fraction.

$\gamma_{sk}$, the specific gravity of the skeleton, g/cm$^3$.

$G$, the coefficient of water saturation, a fraction.

$I_{inc}$, the ice content as ice inclusions, a fraction.

$q$, the peat content (ratio of peat mass to mass of dry soil), a fraction.

$W_u$, the mean content of unfrozen water in the temperature range where significant phase transformations of soil moisture occur, a fraction.

We have

$$\varepsilon_i = \frac{\gamma_{spec}\,(1 + W_t)}{\gamma} - 1, \tag{3}$$

$$\gamma_{sk} = \frac{\gamma}{1 + W_t}, \tag{4}$$

$$G = \frac{W_t \gamma \gamma_{spec}}{[\gamma_{spec}\,(1 + W_t) - \gamma]\,\gamma_w}, \tag{5}$$

and

$$I_{inc} = \frac{\gamma_{spec} W_{inc}}{\gamma_{ice} + \gamma_{spec}\,(W_t - 0.1W_u)}, \tag{6}$$

where $\gamma_w$ is the density of water and $\gamma_{ice}$ is the density of ice, both in g/cm$^3$.

The "temperature range where significant phase transformations of soil moisture occur" is that range in which a 1°C change in temperature produces a change of 1% or more in the quantity of unfrozen water.

Table 1 gives the assumed values for the mean content of unfrozen water $W_u$ in this range, for unsalted soils thawing as the temperature rises.

Table 1.

| Soil type | $W_u$ |
|---|---|
| Sands | 0 |
| Sandy loams | 0.07 |
| Finest clays | 0.08 |
| Clay loams | 0.10 |
| Clays | 0.15 |
| Peats | 1.0 |

For the quantity of unfrozen water in freezing soils, multiply the value of $W_u$ in Table 1 by the constant 1.2.

The peat content of a soil $q$ is computed by the formula

$$q = \frac{A}{\gamma_{spec(p.s.)}} - B, \tag{7}$$

where $\gamma_{spec(p.s.)}$ is the density of the peaty soil, and $A$ (cm$^3$/g) and $B$ (dimensionless) are parameters depending on the type of soil. Table 2 gives the values of $A$ and $B$.

Table 2.

| Type of soil in which peat forms | $A$ | $B$ |
|---|---|---|
| Sandy | 3.46 | 1.30 |
| Clayey | 3.26 | 1.19 |

The tabulation below gives some values for the densities of soil, water and ice, which can be used in most cases.

| Soil | Density, g/cm$^3$ | Substance | Density, g/cm$^3$ |
|---|---|---|---|
| Sand | 2.66 | Water | 1 |
| Sandy loam | 2.70 | Ice | 0.92 |
| Clay loam | 2.71 | | |
| Clay | 2.74 | | |
| Peat | 1.52 | | |

Experimental studies have shown that the thermal diffusivity $a$ and the thermal conductivity $\lambda$ for sands, sandy loams, clay loams, clays and peats can be read as functions of the water saturation $G$ from the graphs of Figs. 7 and 8.

The thermophysical properties of soils are linked by the relation

$$a = \lambda / C_v, \tag{8}$$

where $C_v$ is the volumetric specific heat and $C_{spec}$ is the specific heat.

Table 3 gives thermal conductivities for thawed and completely frozen soils.

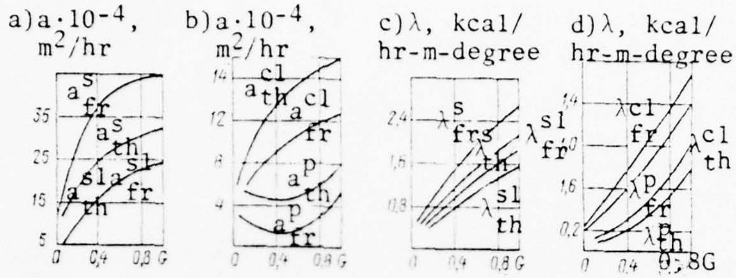


Fig. 7. Thermal diffusivities ($a$) and conductivities ($\lambda$) of soils, as functions of water saturation $G$. (a) Values of $a$ for sands (superscript $s$) and sandy loams (superscript $sl$); (b) values of $a$ for clays (superscript $cl$, including clay loams) and peats (superscript $p$); (c) $\lambda$ for sands and sandy loams; (d) $\lambda$ for clays and peats. Subscripts denote frozen ($fr$) and thawed ($th$) soils.

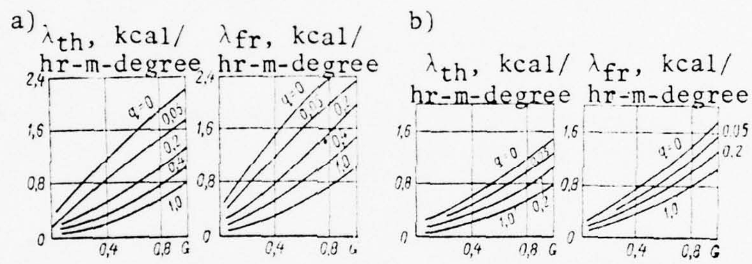


Fig. 8. Thermal conductivities of thawed ($\lambda_{th}$) and frozen ($\lambda_{fr}$) soils as functions of the water saturation $G$ and the peat content $q$. (a) Sandy soils; (b) clayey soils.

The values of $C_{fr}$ in Table 4 apply to a temperature of -10°C. In the temperature range from -0.5°C to -10°C, this quantity is determined as a function of the quantity of unfrozen water at the given temperature, by the formula

$$C'_{fr} = \frac{1}{W_t} [C_{fr} (W_t - W_u) + C_{th} W_u]. \tag{9}$$

Table 4 compares the thermal conductivities of sand $\lambda_{fr}$ as found from Table 3 and from the graphs of Figs. 7 and 8.

For some thermophysical calculations it is necessary to know the specific heats of water and ice; Table 5 gives these values.

Table 3.

| Density $\gamma$, ton/m$^3$ | Total moisture content $W_t$ | Soil thermal conductivity: Sands $\lambda_{th}$ W/m-K | Sands $\lambda_{th}$ kcal/hr-m-degree | Sands $\lambda_{fr}$ W/m-K | Sands $\lambda_{fr}$ kcal/m-hr-degree | Sandy loams $\lambda_{th}$ W/m-K | Sandy loams $\lambda_{th}$ kcal/hr-m-degree | Sandy loams $\lambda_{fr}$ W/m-K | Sandy loams $\lambda_{fr}$ kcal/m-hr-degree | Clay loams & clays $\lambda_{th}$ W/m-K | Clay loams & clays $\lambda_{th}$ kcal/hr-m-degree | Clay loams & clays $\lambda_{fr}$ W/m-K | Clay loams & clays $\lambda_{fr}$ kcal/hr-m-degree | Volumetric specific heat of soil $C_{th}$ kJ/m$^3$-K | $C_{th}$ kcal/m$^3$-degree | $C_{fr}$ kJ/m$^3$-K | $C_{fr}$ kcal/m$^3$-degree |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1.2 | 0.05 | 0.46 | 0.4 | 0.60 | 0.52 | — | — | — | — | — | — | — | — | 1197 | 285 | 1092 | 260 |
| 1.2 | 0.1 | 0.72 | 0.62 | 0.92 | 0.79 | 0.44 | 0.38 | 0.52 | 0.45 | — | — | — | — | 1344 | 320 | 1134 | 270 |
| 1.4 | 0.05 | 0.66 | 0.57 | — | — | — | — | — | — | — | — | — | — | 1386 | 330 | 1260 | 300 |
| 1.4 | 0.1 | 1.01 | 0.87 | 1.25 | 1.08 | 0.60 | 0.52 | 0.80 | 0.69 | 0.51 | 0.44 | 0.79 | 0.68 | 1554 | 370 | 1323 | 315 |
| 1.4 | 0.15 | 1.16 | 1.0 | 1.45 | 1.25 | 0.82 | 0.71 | 1.02 | 0.88 | 0.65 | 0.56 | 0.97 | 0.84 | 1722 | 410 | 1386 | 330 |
| 1.4 | 0.20 | — | — | — | — | 0.97 | 0.84 | 1.22 | 1.05 | 0.75 | 0.65 | 1.09 | 0.94 | 1890 | 450 | 1449 | 345 |
| 1.4 | 0.25 | — | — | — | — | 1.07 | 0.92 | 1.35 | 1.16 | 0.84 | 0.72 | 1.16 | 1.00 | 2058 | 490 | 1512 | 260 |
| 1.6 | 0.05 | 0.87 | 0.75 | 1.06 | 0.91 | — | — | — | — | — | — | — | — | 1696 | 380 | 1428 | 340 |
| 1.6 | 0.1 | 1.22 | 1.05 | 1.57 | 1.35 | — | — | — | — | — | — | — | — | 1806 | 430 | 1512 | 360 |
| 1.6 | 0.15 | 1.45 | 1.25 | 1.86 | 1.6 | 1.08 | 0.93 | 1.28 | 1.1 | 0.84 | 0.72 | 1.14 | 0.98 | 1974 | 470 | 1554 | 370 |
| 1.6 | 0.2 | 1.58 | 1.36 | 2.01 | 1.73 | 1.22 | 1.05 | 1.50 | 1.29 | 1.02 | 0.88 | 1.30 | 1.12 | 2183 | 520 | 1659 | 395 |
| 1.6 | 0.25 | 1.64 | 1.41 | 2.11 | 1.35 | 1.16 | 1.67 | 1.44 | 1.11 | 0.96 | 1.44 | 1.44 | 1.24 | 2373 | 565 | 1722 | 410 |
| 1.6 | 0.3 | — | — | 2.21 | 1.93 | 1.39 | 1.2 | 1.80 | 1.55 | 1.16 | 1.00 | 1.51 | 1.3 | 2562 | 610 | 1806 | 430 |
| 1.6 | 0.35 | — | — | — | — | 1.51 | 1.3 | 1.91 | 1.65 | 1.22 | 1.05 | 1.57 | 1.35 | 2730 | 650 | 1889 | 445 |
| 1.6 | 0.4 | — | — | — | — | — | — | 2.00 | 1.72 | 1.28 | 1.1 | 1.64 | 1.41 | 2940 | 700 | 1953 | 465 |
| 1.6 | 0.6 | — | — | — | — | — | — | — | — | — | — | 1.74 | 1.5 | — | — | 2100 | 500 |
| 1.8 | 0.1 | 1.51 | 1.3 | 1.86 | 1.6 | — | — | — | — | — | — | — | — | 2016 | 480 | 1680 | 400 |
| 1.8 | 0.15 | 1.78 | 1.55 | 2.20 | 1.9 | 1.38 | 1.19 | 1.52 | 1.31 | 1.16 | 1.0 | 1.43 | 1.23 | 2226 | 530 | 1764 | 420 |
| 1.8 | 0.2 | 1.91 | 1.65 | 2.44 | 2.1 | 1.55 | 1.34 | 1.76 | 1.52 | 1.30 | 1.12 | 1.60 | 1.38 | 2136 | 580 | 1848 | 440 |
| 1.8 | 0.25 | 2.03 | 1.75 | 2.59 | 2.23 | 1.66 | 1.43 | 1.97 | 1.7 | 1.44 | 1.24 | 1.77 | 1.53 | 2688 | 640 | 1932 | 460 |
| 1.8 | 0.3 | — | — | 2.69 | 2.32 | 1.72 | 1.48 | 2.11 | 1.82 | 1.49 | 1.28 | 1.87 | 1.61 | 2898 | 690 | 2016 | 480 |
| 1.8 | 0.35 | — | — | — | — | 1.75 | 1.51 | 2.24 | 1.93 | 1.51 | 1.33 | 1.93 | 1.66 | 3108 | 740 | 2100 | 500 |
| 1.8 | 0.4 | — | — | — | — | — | — | 2.32 | 2.0 | 1.62 | 1.4 | 1.99 | 1.72 | 3339 | 795 | 2184 | 520 |
| 1.3 | 0.6 | — | — | — | — | — | — | — | — | — | — | 2.09 | 1.8 | — | — | 2352 | 560 |
| 2.0 | 0.15 | 2.04 | 1.76 | 2.55 | 2.2 | 1.62 | 1.4 | 1.74 | 1.5 | — | — | — | — | 2478 | 590 | 1974 | 470 |
| 2.0 | 0.2 | 2.32 | 2.0 | 2.91 | 2.42 | 1.81 | 1.56 | 2.03 | 1.75 | 1.44 | 1.24 | 1.74 | 1.5 | 2730 | 650 | 2058 | 490 |
| 2.0 | 0.25 | 2.62 | 2.26 | 3.16 | 2.72 | 2.01 | 1.73 | 2.24 | 1.93 | 1.57 | 1.35 | 1.91 | 1.65 | 2961 | 705 | 2142 | 510 |
| 2.0 | 0.2 | — | — | — | — | 2.10 | 1.8 | 2.44 | 2.1 | 1.67 | 1.44 | 2.03 | 1.75 | 3234 | 770 | 2226 | 530 |
| 2.0 | 0.35 | — | — | — | — | — | — | — | — | 1.78 | 1.53 | 2.16 | 1.86 | 3444 | 820 | 2331 | 555 |

Table 4.

| Density of soil $\gamma$, $kg/m^3$ | Total moisture content $W_t$, % | Water saturation G | $\lambda_{fr}$ from Table 3 | | $\lambda_{fr}$ from graph | |
|---|---|---|---|---|---|---|
| | | | W/m-K | kcal/hr-m-degree | W/m-K | kcal/hr-m-degree |
| 16000 | 0.10 | 0.32 | 1.57 | 1.35 | 1.51 | 1.30 |
| 18000 | 0.15 | 0.44 | 1.86 | 1.60 | 1.88 | 1.62 |
| 18000 | 0.10 | 0.43 | 1.86 | 1.60 | 1.86 | 1.60 |
| 18000 | 0.15 | 0.58 | 2.20 | 1.90 | 2.2 | 1.90 |
| 18000 | 0.20 | 0.71 | 2.44 | 2.10 | 2.55 | 2.20 |

Table 5.

| Tamperature | | Specific heat of water $C_{spec}$ | | Temperature | | Specific heat or ice $C_{spec}$ | |
|---|---|---|---|---|---|---|---|
| K | +°C | kJ/kg-K | kcal/kg-deg | K | -°C | kJ/kg-K | kcal/kg/deg. |
| 273 | 0 | 4,2 | 1.010 | 273 | 0 | 2 125 | 0.506 |
| 274 | 1 | 4,238 | 1.009 | 272 | 1 | 2.117 | 0.504 |
| 275 | 2 | 4.234 | 1.008 | 271 | 2 | 2.108 | 0.502 |
| 276 | 3 | 4.234 | 1.008 | 269 | 3 | 2.100 | 0.500 |
| 277 | 4 | 4.229 | 1.007 | 268 | 4 | 2,092 | 0.498 |
| 278 | 5 | 4.225 | 1.006 | 267 | 5 | 2.083 | 0.496 |
| 279 | 6 | 4.221 | 1.005 | 266 | 6 | 2.079 | 0.495 |
| 280 | 7 | 4,217 | 1.004 | 265 | 7 | 2.071 | 0.493 |
| 281 | 8 | 4.217 | 1.004 | 264 | 8 | 2.062 | 0,491 |
| 282 | 9 | 4,217 | 1.004 | 263 | 9 | 2.054 | 0,489 |
| 283 | 10 | 4.213 | 1.003 | 262 | 10 | 2.045 | 0.487 |
| 284 | 11 | 4.213 | 1.003 | 261 | 11 | 2.037 | 0,485 |
| 285 | 12 | 4.208 | 1,002 | 260 | 12 | 2,029 | 0.483 |
| 286 | 13 | 4.208 | 1.002 | 259 | 13 | 2.020 | 0.481 |
| 287 | 14 | 4.208 | 1.002 | 258 | 14 | 2.016 | 0.480 |
| 288 | 15 | 4.204 | 1,001 | 257 | 15 | 2.008 | 0.478 |
| 289 | 16 | 4.204 | 1.001 | 256 | 16 | 2,008 | 0.476 |
| 290 | 17 | 4.204 | 1.001 | 255 | 17 | 1.991 | 0.474 |
| 291 | 18 | 4.200 | 1.000 | 254 | 18 | 1.982 | 0.472 |
| 292 | 19 | 4.200 | 1.000 | 253 | 19 | 1.974 | 0.470 |
| 293 | 20 | 4,200 | 1,000 | 252 | 20 | 1.966 | 0.468 |
| 294 | 21 | 4.200 | 1.000 | 251 | 21 | 1.961 | 0.467 |
| 295 | 22 | 4.200 | 1,000 | 250 | 22 | 1,957 | 0.466 |
| 296 | 23 | 4.196 | 0,999 | 249 | 23 | 1.945 | 0.463 |
| 297 | 24 | 4.196 | 0.999 | 248 | 24 | 1.936 | 0.461 |
| 298 | 25 | 4.196 | 0.999 | 247 | 25 | 1,927 | 0.459 |
| 299 | 26 | 4.196 | 0.999 | 246 | 26 | 1,919 | 0.457 |
| 300 | 27 | 4.196 | 0.999 | 245 | 27 | 1.911 | 0.455 |
| 301 | 28 | 4,196 | 0.999 | 244 | 28 | 1.907 | 0.454 |
| 302 | 29 | 4.196 | 0.999 | 243 | 29 | 1.898 | 0.452 |

*Compressive, tensile and shear strengths of frozen soils*

In zones of stress concentration where mineral particles in frozen soil make contact, the stresses are greatly magnified. The result is that the ice melts and the water moves to regions of lower stress. Thus the quantity of unfrozen water in a frozen soil changes under stress, so that the strength of the bond between soil particles and ice is altered.

The mechanical properties of freezing, frozen and thawing soils depend on the temperature, the stress state in the region under consideration, and the time over which the load acts.

Table 6.

| Soil temperature | | Prescribed (normative) resistance of frozen soil to normal pressure, $R_n$ | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | coarse-fragmented & sandy, coarse & med. coarse | | sandy, fine and silty | | sandy loams including silty | | clay loams & clays inc. silty | | peat, massive, with cryogenic texture | | All types of soils with ice interlayers & inclusions $0.2 \leq I_{inc} \leq 0.4$ | |
| K | °C | MPa | kg/cm² | MPa | kg/cm² | MPa | kg/cm² | MPa | kg/cm² | MPa | kg/cm² | MPa | kg/cm² |
| 272.7 | −0.5 | 0.9 | 9 | 0.7 | 7 | 0.5 | 5 | 0.4 | 4 | — | — | 0.3 | 3 |
| 272.2 | −1.0 | 1.2 | 12 | 0.9 | 9 | 0.7 | 7 | 0.6 | 6 | — | — | 0.4 | 4 |
| 271.7 | −1.5 | 1.4 | 14 | 1.1 | 11 | 0.9 | 9 | 0.7 | 7 | — | — | 0.5 | 5 |
| 271.2 | −2.0 | 1.6 | 16 | 1.3 | 13 | 1.0 | 10 | 0.8 | 8 | — | — | 0.6 | 6 |
| 270.7 | −2.5 | 1.8 | 18 | 1.4 | 14 | 1.1 | 11 | 0.9 | 9 | — | — | 0.7 | 7 |
| 270.2 | −3.0 | 1.9 | 19 | 1.6 | 16 | 1.3 | 13 | 1.0 | 10 | 0.8 | 8 | 0.8 | 8 |
| 269.7 | −3.5 | 2.1 | 21 | 1.7 | 17 | 1.4 | 14 | 1.1 | 11 | 0.9 | 9 | 0.8 | 8 |
| 269.2 | −4.0 and below | 2.3 | 23 | 1.8 | 18 | 1.5 | 15 | 1.2 | 12 | 1.0 | 10 | 0.9 | 9 |

Notes:

1. Identification of soils: (*1*) Coarse-fragmented and sandy, coarse and moderately coarse; (*2*) Sandy, fine and silty; (*3*) Sandy loams, including silty; (*4*) Clay loams and clays, including silty; (*5*) Peat, massive, with cryogenic texture; (*6*) All soils with ice interlayers and ice inclusions with $0.2 \leq I_{inc} \leq 0.4$.

2. $R^n$ values given for soils with $I_{inc} < 0.2$.

3. For soils of type (*6*), sandy footings with a thickness of $h \geq 0.2$ m should be provided.

4. Prescribed (normative) strengths of soils with $I_{inc} > 0.4$ are taken from results of special studies.

(Notes continued on next page)

(Notes to Table 6 - Continued)

5. For short-term loads the $R^n$ values may be multiplied by 1.5 (load acting for 0.5 hr), 1.4 (1 hr), 1.3 (2 hr), 1.2 (8 hr), or 1.1 (24 hr) [Translator's note: Original had 2.4 hr]. These increases can be made provided the loads are removed for times at least as long as they are applied.

6. The $R^n$ values may be corrected on the basis of construction experience or soil tests.

7. The prescribed (normative) strengths do not apply to frozen soils with salt contents greater than 25%; for these soils, special tests should be run to determine the strengths.

Table 7.

| Soil temperature | | Prescribed (normative) shear strengths for soil against soil, $R^n_{sh}$ | | | |
|---|---|---|---|---|---|
| | | Sandy, all varieties | | Clayey, including silty | |
| K | °C | MPa | kg/cm² | MPa | kg/cm² |
| 272.7 | 0.5 | 0.12 | 1.2 | 0.08 | 0.8 |
| 272.3 | 1 | 0.17 | 1.7 | 0.12 | 1.2 |
| 271.7 | 1.5 | 0.21 | 2.1 | 0.15 | 1.5 |
| 271.2 | 2 | 0.24 | 2.4 | 0.17 | 1.7 |
| 270.7 | 2.5 | 0.27 | 2.7 | 0.19 | 1.9 |
| 270.2 | 3 | 0.30 | 3.0 | 0.21 | 2.1 |
| 269.7 | 3.5 | 0.32 | 3.2 | 0.23 | 2.3 |
| | 4 and below | 0.34 | 3.4 | 0.25 | 2.5 |

Tables 6, 7, 8, 9 and 11 present the following data:

(1) Prescribed (normative) normal compressive strengths of frozen soils at various temperatures.

(2) Shear strengths of frozen soils.

(3) Instantaneous and long-term limiting cohesions of frozen soils.

(4) Shear strengths of frozen soils against wood and concrete surfaces.

(5) Normal modulus of elasticity and Poisson coefficient.

Table 11 gives the moduli of normal elasticity $E$ for various soils under various stresses (Figs. 9 and 10).

Table 8.

| Soil | $W_t$, % | Instantaneous and long-term limiting cohesion of permafrosts with undisturbed structure | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | $C_{inst}$ | | $C_{lt}$ | | $C_{inst}$ | | $C_{lt}$ | | $C_{inst}$ | | $C_{lt}$ | |
| | | MPa | kg/cm$^2$ | MPa | kg/cm$^2$ | MPa | kg/cm$^2$ | MPa | kg/cm$^2$ | MPa | kg/cm$^2$ | MPa | kg/cm$^2$ |
| | | 272.9-272.8 K | | | | 272.2-272 K | | | | 269.2-269 K | | | |
| | | from -0.3 to -0.4°C | | | | from -1.0 to -1.2°C | | | | from -4.0 to -4.2°C | | | |
| Banded clay (mineral interlayers) . . . . . . | 30-40 | 0.57 | 5.7 | 0.18 | 1.8 | -- | -- | 0.26 | 2.6 | 1.6 | 16.0 | 0.42 | 4.2 |
| Clay loam, heavy, silty . . . . . | 36 | 0.43 | 4.3 | 0.06 | 0.6 | 0.7 | 7.0 | 0.1 | 1.0 | 1.2 | 12.0 | -- | -- |
| Clay loam, light, silty . . . . . | 30 | 0.41 | 4.1 | 0.09 | 0.9 | 0.74 | 7.4 | -- | -- | 1.1 | 11.0 | 0.2 | 2.0 |
| Sandy loam, heavy, silty . . . . . | 28-34 | 0.4- -0.45 | 4.0- -4.5 | 0.09- -.10 | 0.9- -1.0 | 0.73 | 7.3 | 0.16 | 1.6 | 0.8- -1.5 | 8.0- -15.0 | 0.28- -0.32 | 2.8- -3.2 |
| Sandy loam, heavy, silty, high moisture content . . | 43 | 0.6 | 6.0 | 0.075 | 0.75 | -- | -- | -- | -- | 1.1 | 11.0 | 0.2 | 2.0 |
| The same, with peat content . . . . | 30 | -- | -- | -- | -- | -- | -- | -- | -- | 0.9 | 9.0 | 0.2 | 2.0 |
| Sand, silty . . . | 23 | 1.1 | 11.0 | 0.21 | 2.1 | 1.4 | 14.0 | 0.27 | 2.7 | 2.0 | 20.0 | 0.37- -0.45 | 3.7- -4.5 |

Table 9.

| Soil temperature | | Prescribed (normative) shear strength of frozen soils against concrete and wood footing surfaces | | | |
|---|---|---|---|---|---|
| | | Sandy, all varieties | | Clayey, including silty | |
| K | -°C | MPa | kg/cm$^2$ | MPa | kg/cm$^2$ |
| 272.7 | 0.5 | 0.08 | 0.8 | 0.05 | 0.5 |
| 272.2 | 1 | 1.13 | 1.3 | 0.1 | 1.0 |
| 271.7 | 1.5 | 0.16 | 1.6 | 0.13 | 1.3 |
| 271.2 | 2 | 0.20 | 2.0 | 0.15 | 1.5 |
| 270.7 | 2.5 | 0.23 | 2.3 | 0.18 | 1.8 |
| 270.2 | 3 | 0.26 | 2.6 | 0.2 | 2.0 |
| 269.7 | 3.5 | 0.29 | 2.9 | 0.23 | 2.3 |
| 269.2 | 4 and below | 0.33 | 3.3 | 0.25 | 2.5 |

1. $R^{n}_{sh}$ values should be multiplied by 0.7 when the soils adfreeze to metal surfaces not specially treated.

2. $R^{n}_{sh}$ values may be corrected on the basis of construction experience or soil tests.

3. The prescribed (normative) strengths do not apply to frozen soils with salt contents greater than 25%; for these soils, special tests should be run to determine the strengths.

Table 10.

| Soil | $W_t$, % | Temperature | | Pressure | | Poisson coefficient $\mu$ |
|---|---|---|---|---|---|---|
| | | K | -°C | MPa | kg/cm$^2$ | |
| Frozen sand | 19 | 273.0 | 0.2 | 0.2 | 2 | 0.4 |
| | 19 | 272.3 | 0.8 | 0.6 | 6 | 0.13 |
| Frozen silty clay loam | 28 | 272.9 | 0.3 | 0.15 | 1.5 | 0.35 |
| | 28 | 272.4 | 0.8 | 0.2 | 2 | 0.18 |
| | 25.3 | 271.5 | 1.5 | 0.2 | 2 | 0.14 |
| | 28.7 | 269.2 | 4.0 | 0.6 | 6 | 0.13 |
| Frozen clay | 50.1 | 272.7 | 0.5 | 0.2 | 2 | 0.45 |
| | 53.4 | 271.5 | 1.7 | 0.4 | 4 | 0.35 |
| | 54.8 | 268.2 | 5.0 | 1.2 | 12 | 0.26 |

Table 11.

| Entry No. | Physical model of soil layer | Stress | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 0.1 MPa (1 kg/cm²) | | | | 0.2 MPa (2 kg/cm²) | | | | 0.3 MPa (3 kg/cm²) | | | |
| | | *t* | | *E* | | *t* | | *E* | | *t* | | *E* | |
| | | K | -°C | MPa | kg/cm² | K | -°C | MPa | kg/cm² | K | -°C | MPa | kg/cm² |
| I | Layered . . . . . . . . . | 271.8 | 1.4 | 1430 | 14300 | 271.8 | 1,4 | 1270 | 12700 | — | — | — | — |
| II | » . . . . . . . . . . | 271.5 | 1.7 | 3350 | 33500 | 271.5 | 1,7 | 3010 | 30100 | 271.5 | 1.7 | 2930 | 29300 |
| III | » . . . . . . . . . . | 271.6 | 1.6 | 850 | 8500 | 271.6 | 1,6 | 820 | 8200 | — | — | — | — |
| | Homogeneous: | | | | | | | | | | | | |
| IV | sand . . . . . . . . | — | — | — | — | 271.5 | 1,7 | 4080 | 40700 | — | — | — | — |
| V | clay . . . . . . . . | — | — | — | — | 271.5 | 1,7 | 890 | 8900 | — | — | — | — |

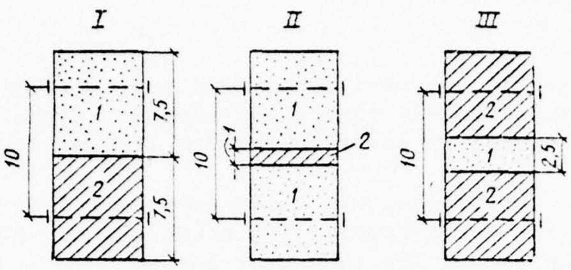


Fig. 9. Arrangements of soil layers in the study of elastic deformations of frozen-soil samples. I, II and III are layer models of the soil structure; 1 represents sand; 2, clay.

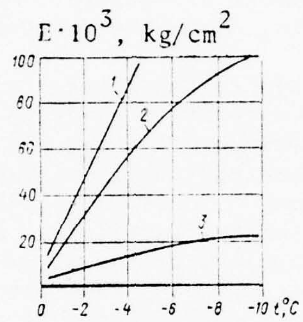


Fig. 10. Modulus of normal elasticity of frozen soils $E$ versus the below-freezing temperature $t$, for an external pressure of $\sigma$ = 2 kgf/cm$^2$. (1) Frozen sand; (2) frozen silty soil; (3) frozen clay.

Table 10 presents values of the Poisson coefficient $\mu$ for frozen soils.

Appendix 2.
Thermotechnical calculations
for the dam embankment and foundation
and the reservoir bottom

Calculations of temperature regimes for dams and spillways on permafrost foundations constitute the most important element in the planning of hydrotechnical structures in water-management systems in the Far North.

Temperature-regime calculations are quite important for justifying the type of structure, and for computing the stability and strength of the structure as a whole and of its component parts.

The best-developed methods are currently those for thermal calculations of non-filter frozen earth dams with frozen central parts. Research is under way into the temperature regimes of thawed filter dams on frozen foundations that thaw as a result of seepage. Methods of calculating the temperature regimes of rockfill dams have received less development.

On the basis of theoretical and experimental studies, Soviet scientists have worked out and put into widespread practice engineering methods for predicting the temperature regimes of frozen and thawed dams and spillways. Exact analytic solutions have been obtained for some of the most important computational schemes. These solutions make it possible to predict the limiting (steady-state) temperature state in non-filter earth dams, and also to solve the problem of how a non-steady-state temperature regime forms in a filter dam. Soviet scientists have also proposed methods for approximate thermal calculations of thawed and frozen earth dams, using the finite-differences approximation for the differential equations of heat exchange in pervious (Fourier-Kirchhoff equation) and impervious soils (Fourier equation). Several theoretical points have formed the subject of research on numerical methods for the computer solution of non-steady-state temperature-regime problems in earth dams and soil foundations.

The practical methods set out in this Appendix for approximate thermotechnical calculations of earth hydrotechnical structures and their permafrost foundations do not exclude the use, when necessary, of the more exact and complex solutions, or of digital or analog computers.

Regardless of the height of the dam and the cryogenic and geological conditions in the foundation, a series of thermotechnical calculations must be performed. These include:

(1) Analysis of the dynamics of thawing of the reservoir bottom.

(2) Construction of an outline of the limiting thaw basin in the permafrosts making up the reservoir bottom. Analysis of the steady-state (limiting) temperature state of the bottom permafrosts in the zone to which the thermal effects of the reservoir and structures of the water-engineering system extend.

(3) Analysis of the dynamics of thawing of the frozen embankment of a frozen dam and of the permafrosts in the bank slopes after the reservoir is filled with water.

(4) Calculation of the steady-state (limiting) temperature regime of the frozen dam, in particular a determination of the position of the zero-degree (Celsius) isotherm, which is the boundary between the thawed (deformable) and frozen (stable) zones of the dam cross-section.

(5) Calculation of the dynamics of growth of the frozen curtain in a dam with freezing columns. This includes a determination of the mean temperature of a frozen cutoff of finite thickness, the velocities of air in the columns, and the capacity of the blowers.

Several approximate methods for thermotechnical calculations appear below. These methods are recommended for practical use in the planning of dams up to 25-30 m high.

Appendix 2 presents empirical relations for thermotechnical calculations of the embankment and foundation of an earth dam, based on experimental and theoretical knowledge.

All the relations are fairly simple and find good confirmation in field data.

It does not appear possible to convert the empirical relations in Appendix 2 to SI units. The results of the converted relations would not have practical value, and the formulas would become unusable. The values obtained in final form from the empirical relations may be converted to SI units.

## 1. Dynamics of thawing of permafrost under the reservoir bottom (one-dimensional problem)

When reservoirs are created in permafrost regions, it is important to know to what depth and how fast the soils under the reservoir bottom will thaw. The dimensions of the thawed zone under the reservoir make it possible to determine the parameters of the seepage flow generated in the thawing layers of the foundation. Equation (10) should be used to determine the dynamics of motion of the thawing boundary in the reservoir bottom (without allowance for seepage in the thawing period).

The portion of the reservoir under consideration is assumed to lie at a considerable distance from the dam embankment and reservoir banks. Therefore it is assumed that air temperature does not directly affect the soil regime in the section under consideration, and that the bottom thaws in a vertical direction (one-dimensional problem).

At a time $\tau$ (hr) after the filling of the reservoir, the soil has thawed to a depth $x$:

$$x = \sqrt{\frac{2\lambda_{th} t_1 \tau}{0.9\rho W_t + C_{fr} t_2}}, \tag{10}$$

where $x$ is the depth from the surface of the reservoir bottom to the thawing boundary of the frozen soil (zero-degree [Celsius] isotherm), m (see Fig. 11); $\lambda_{th}$ is the thermal conductivity of the thawed soil, kcal/hr-m-degree; $t_1$ is the water temperature in the reservoir at the level of the soil surface, °C; $t_2$ is the mean initial temperature of the frozen soil before the filling of the reservoir, °C; $\rho$ is the latent heat of the phase transformation of soil moisture (80,000 kcal/ton ["ton" always denotes metric ton]); $W_t$ is the total content of moisture or ice, a fraction; $C_{fr}$ is the volumetric specific heat of the frozen soil, kcal/m$^3$-degree; and $\tau$ is the time from the filling of the reservoir to the moment under consideration, hr.

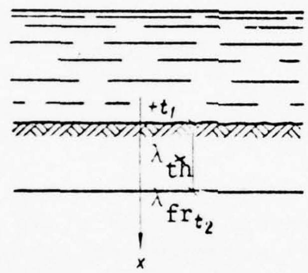


Fig. 11. Definition of the thawing boundary of frozen soil under the reservoir bottom.

The quantity of heat, in kcal/m$^3$, needed to thaw the soil can be determined by the formula

$$Q = 0.9\rho W_t + C_{fr} t_2. \tag{11}$$

Appendix 3 gives a sample calculation of the thawing boundary in a reservoir bottom, using Eq. (10).

The limiting depth of thawing of the bottom in the center of a reservoir with width $B$, meters, can also be found, with accuracy enough for practical purposes, by the formula

$$x = 0.5B \cot \left(\frac{\pi}{2} \frac{\lambda_{th} t_0}{\lambda_{th} t_0 - \lambda_{fr} t_1}\right), \tag{12}$$

where $x$ is the depth of thawing, m; $B$ is the width of the reservoir in the region where it is assumed to affect the dam thermally, m; $t_0$ is the temperature of the permafrost at the level of zero amplitude (initial soil temperature); and $t_1$ is the water temperature, °C.

The ratio of reservoir length $L$ to width $B$ has little effect on the $x$ value found by Eq. (12) (see Appendix 3, Example 1).

## 2. Steady-state soil temperatures in the reservoir bottom and dam foundation

The analytical calculation of the three-dimensional steady-state temperature field in the reservoir bottom and in the foundation of a frozen dam should be carried out by Eq. (13):

$$u=\frac{1}{\pi}\left\{\arctan\frac{x^2+(y-0{,}5B)^2+(x-y+0{,}5B)\left[z+\sqrt{x^2+(y-0{,}5B)^2+z^2}\right]}{\left[z+\sqrt{x^2+(y-0{,}5B)^2+z^2}\right]^2+(x-y+0{,}5B)\left[z+\sqrt{x^2+(y-0{,}5B)^2+z^2}\right]}\right.$$

$$+\arctan\frac{x^2+(y+0{,}5B)^2+(x+y+0{,}5B)\left[z+\sqrt{x^2+(y+0{,}5B)^2+z^2}\right]}{\left[z+\sqrt{x^2+(y+0{,}5B)^2+z^2}\right]^2+(x+y+0{,}5B)\left[z+\sqrt{x^2+(y+0{,}5B)^2+z^2}\right]} \qquad (13)$$

$$\left.-\arctan\frac{x}{z}\right\}.$$

The symbols in Eq. (13) are defined in Fig. 12.

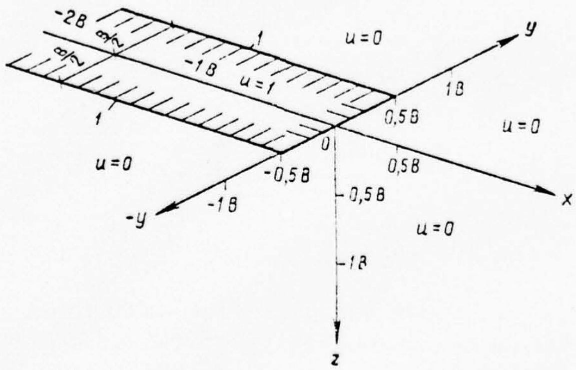


Fig. 12. Diagram for calculation of the limiting thawing of a reservoir bottom and the foundation of a frozen dam. $B$ is the width of the dam; $u$ is the relative (dimensionless) temperature; and $1$ marks the reservoir outline.

In Eq. (13), the relative (dimensionless) temperature $u$ is linked with the actual temperature value $t$ by the following relations, which allow for the differing thermophysical characteristics of the soil in thawed and frozen states:

(a) For temperatures below freezing, $t \leq 0$ (frozen-soil zone),

$$U = \frac{\lambda_{fr}(t + t_2)}{\lambda_{fr} t_2 + \lambda_{th} t_1}; \tag{14}$$

(b) for temperatures above freezing, $t \geq 0$ (thawed-soil zone),

$$U = \frac{\lambda_{fr} t_2 + \lambda_{th} t}{\lambda_{fr} t_2 + \lambda_{th} t_1}, \tag{15}$$

where $\lambda_{fr}$ and $\lambda_{th}$ are the thermal conductivities of the soil in the frozen and thawed states respectively, kcal/hr-m-degree; $t_1$ is the constant, long-term mean water temperature in the bottom layers of the reservoir; and $t_2$ is the absolute mean annual below-freezing temperature of the soil surface that comes into contact with air, °C.

The width of the reservoir $B$ is taken as the unit of length, so that the temperature cross-section becomes compact and the computations are simplified. The relative temperature of the underwater soil surface $u_w = 1$; the temperature of the "dry" surface $u_n = 0$.

The relative $u$ scale of temperature is constructed on an arbitrary zero, equal to the temperature of the soil surface that makes contact with the air.

The boundary conditions for the computational scheme are written in the following form:

$$U = 1 \text{ for } x \leq 0 \text{ and } -\frac{B}{2} \leq y \leq \frac{B}{2};$$

$$U = 0 \text{ for } -\infty < x < \infty \text{ and } -\frac{B}{2} \geq y \geq \frac{B}{2};$$

$$U = 0 \text{ for } x \geq 0 \text{ and } -\infty < y < +\infty.$$

Equation (13) was derived under the following additional conditions, which have to be taken into account in practical calculations for concrete problems:

Water level is assumed constant at the annual mean level.

The temperatures of wetted and dry surfaces are also constant and equal to the mean annual values.

The width of the reservoir $B$ is constant over the whole length, which far exceeds the width.

The soils in the dam embankment and reservoir bed are uniform and identical in thermophysical properties.

The width of the river valley far exceeds the transverse dimensions (width of cross-section) of the dam; the valley relief is quite gentle and weakly expressed; the dam cross-section is rather flattened; and the formula does not allow for relief features or the height or outline of the dam cross-section.

There is no water in the lower pool.

The formula does not allow for the effect of heat from the interior of the earth.

These conditions, which do not make the theoretical treatment using Eq. (13) any less valuable, do somewhat limit the chances for practical use of the treatment. In particular, when the thermophysical properties of a multilayer soil structure are averaged and the structure is equated to an arbitrary uniform soil, inevitably some error is introduced, and this lowers the accuracy of the computations by Eq. (13) for real conditions of a frozen soil.

The shapes of the isotherms in the real dam cross-section will also differ from the design shapes as worked out on the half-plane at the zero level of the dam. The other assumptions have less influence on the accuracy of results from Eq. (13). *This relation is now the only exact analytic* solution of the three-dimensional problem of the steady-state temperature distribution in the foundation of the dam-reservoir complex. This equation can yield approximate computational methods that take account of the geometric parameters of the dam and the local relief.

For the two-dimensional problem of constructing a plane, steady-state temperature field under a reservoir on a permafrost foundation, the calculations can be performed by the equation

$$t(x,y) = \frac{1}{\pi}\left(\frac{\lambda_{th}}{\lambda_{fr}} t_w - t_d\right)\left(\arctan\frac{\frac{B}{2} - x}{y} + \arctan\frac{\frac{B}{2} + x}{y}\right) + t_d + G^g_y, \tag{16}$$

where $t(x,y)$ is the temperature at some point of the freeze-thaw zone of the reservoir-bottom soil; $t_w$ is the given mean annual temperature of the underwater soil surface, °C; $t_d$ is the long-term mean natural temperature of the frozen soil, averaged over depth from the base of the active layer to the level of zero amplitude (usually to a depth of 15-20 m); and $G^g_y$ is the geometric gradient, in degrees per meter, taken from reference literature on the region of the planned dam (in most permafrost regions $G$ = 0.02-0.03 degree/m, and this may be left out of account in calculations of bottom thawing to a depth of 10-15 m).

Figure 13 shows the computational scheme and the position of the zero-degree (Celsius) isotherm, which limits the thaw basin under the reservoir.

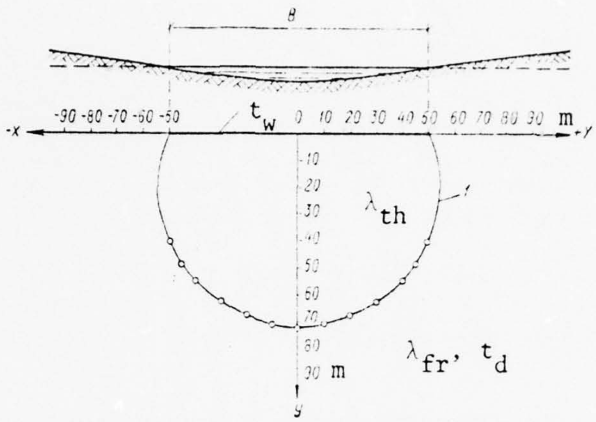


Fig. 13. Computational scheme for determining the limiting thaw basin under the bottom of a reservoir, for $L \geq$ 2-3 $B$. Curve 1 is the calculated thawing contour, on which $t(x,y) = 0$ by Eq. (16).

Equation (16) allows for phase transformations of water by reducing the nonuniform medium of thawing and frozen soil into a uniform medium, the "reduced medium," which is arbitrarily assumed to consist just of frozen soil. The frozen-soil temperatures and the thaw-basin outline, which are of interest to us from the standpoint of dam and foundation stability, are not distorted. The soil temperature inside the thaw basin decreases by a factor of $\lambda_{th}/\lambda_{fr}$ when a "thermal stamp" with temperature $t_w$ is imposed over the reservoir bottom. The stamp is taken to be a reservoir with width $B$. When Eq. (16) is used to find the temperature of the reservoir bottom, the outline of the natural underwater banks of the reservoir and the relief of the "dry" surface are not taken into account. The accuracy of the calculation is less strongly affected by such simplifying steps as averaging the thermal conductivities of the soils, reducing the thawing and frozen medium to an arbitrary uniform frozen medium, and the assumption that the rectangular shape of the reservoir in plan (reservoir length $L$) satisfies the inequality $L \geq$ (2-3) $B$.

Let us neglect the effect of the geothermal gradient and take the width of the reservoir near the dam as constant and equal to $B$, with the reservoir length $L >> B$. Then at any point in the region of the steady-state temperature field where the bottom soils are being heated, the temperature can be determined by the graph-analytical method of constructing a steady-state temperature field in the base of a flat stamp (two-dimensional problem).

The temperature at any point ($x$, $y$) of the foundation is computed by the formula

$$t(x, y) = \left(\frac{\lambda_{th}}{\lambda_{fr}} t_w - t_0\right) \frac{\Omega}{\pi} + t_0, \tag{17}$$

where the angle of opening

$$\Omega = \arctan \frac{0.5B - x}{y} + \arctan \frac{0.5B + x}{y} \text{ (in radians)}$$

can be determined by graphic means (Fig. 14); $t_w$ is the temperature of the reservoir-bottom surface, taken equal to the water temperature in the bottom layer; $t_0$ is the mean permafrost temperature at the level of zero amplitude (with a minus sign in both terms of the equation); and $\frac{\lambda_{th}}{\lambda_{fr}} t_w - t_0$ is the complete temperature difference $\Delta t$, where $t_0$ is taken as the arbitrary zero. For example, if $t_0 = -4°C$ and $\frac{\lambda_{th}}{\lambda_{fr}} t_w = \frac{1.35}{1.65} \cdot 4 = 3.3°C$, then $\Delta t = 7.3°C$.

The temperature $t(x, y)$ characterizes the steady-state (limiting) temperature condition in the reservoir bottom.

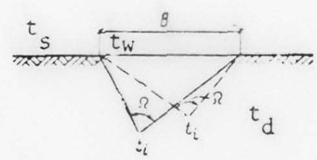


Fig. 14. Computational scheme for determining the limiting thawing zone by using angles of opening $\Omega$.

In the center of the reservoir, the maximum thawing can be found by the formula

$$x = 0.5B \cot \left[\frac{\pi}{2} \frac{\lambda_{th} t_0}{\lambda_{th} t_0 - \lambda_{fr} t_1}\right]. \tag{18}$$

3. Dynamics of thawing of a frozen bank slope after the filling of the reservoir

The dynamics (rate and depth) of thawing of a completely frozen permafrost massif in a reservoir-bank slope (or, to an approximation, in the upstream slope of a dam) is determined by the following formulas (see Fig. 15):

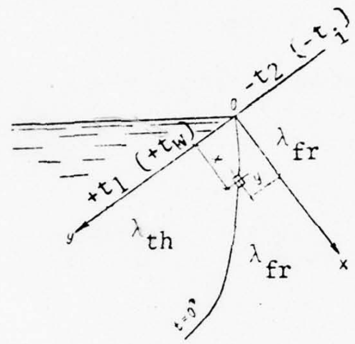


Fig. 15. Computational scheme for the thawing of a soil massif placed on an incline in the reservoir zone.

$$y = x\sqrt{\frac{\lambda_{fr} t_2}{\lambda_{th} t_1 - \frac{Q}{2\tau} x^2} - 1}\ ; \tag{19}$$

$$y = -x\sqrt{\frac{\lambda_{th} t_1}{\lambda_{fr} t_2 + \frac{Q}{2\tau} x^2} - 1}\ . \tag{20}$$

If $\lambda_{fr} t_2 \geq \lambda_{th} t_1$, the boundary of the frozen zone is found by Eq. (19) alone. If $\lambda_{fr} t_2 < \lambda_{th} t_1$, both equations are used. In this case, Eq. (19) applies when $x^2 \geq \frac{\lambda_{th} t_1 - \lambda_{fr} t_2}{Q/2\tau}$, and Eq. (20) when $x^2 < \frac{\lambda_{th} t_1 - \lambda_{fr} t_2}{Q/2\tau}$.

The notation in Eqs. (19) and (20) is the same as in Eq. (10), with the following addition:

> $t_2$, the mean annual temperature of the embankment soil surface. (In approximate calculations it can be assumed equal to the mean annual outdoor-air temperature.)

For $\tau = \infty$, Eq. (19) takes on the following form:

$$y = x\sqrt{\frac{\lambda_{fr} t_2}{\lambda_{th} t_1} - 1}\ . \tag{21}$$

That is, the boundary of the frozen zone will be a straight line.

The boundary of the frozen zone under the reservoir bottom is a special case of Eq. (19).

For $y = \infty$, Eq. (10) takes the form

$$x\sqrt{\frac{\lambda_{fr} t_2}{\lambda_{th} t_1 - \frac{Q}{2\tau} x^2} - 1} = \infty.$$

The quantities $x$, $\lambda_{fr}$ and $t_2$ in the numerator are all finite, so that a necessary condition for the equality is

$$\lambda_{th} t_1 - \frac{Q}{2\tau} x^2 = 0.$$

Solving this equation for $x$, we obtain $x = \sqrt{2\lambda_{th} t_1 \tau / Q}$.

Appendix 3 (Example 3) shows a determination of the dynamics of the soil-thawing boundary in a frozen slope when a reservoir is created.

4. Calculation of the steady-state temperature regime of a frozen dam frozen through before the filling of the reservoir, and determination of the steady-state (limiting) position of the zero-degree (Celsius) isotherm

Under consideration is an earth dam of unbounded height (infinite shell); see Fig. 16. Given a long-term mean temperature $t_1$ above freezing at the surface of the upstream slope below the normal pool elevation, and a long-term mean outdoor-air temperature $t_2$ below freezing above the waterline and on the downstream slope.

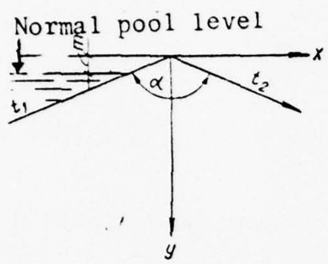


Fig. 16. Computational scheme for determining the temperature field of a symmetrical homogeneous dam under steady-state conditions.

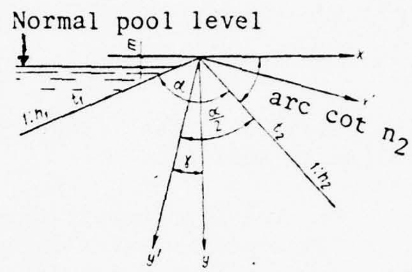


Fig. 17. Computational scheme for determining the temperature field of an asymmetrical homogeneous dam under steady-state conditions.

The problem is solved by the method of conformal mappings.

The formula recommended for determining the temperature at any point of the dam is

$$\left.\begin{aligned} t_w &= t_2 + \frac{t_1 - t_2}{\pi} \arctan \frac{e^A}{\sqrt{\cot^2 B + 1} - e^A \cot B}; \\ A &= \frac{\pi}{\alpha} \ln \frac{1}{m} \sqrt{\frac{x^2 + y^2}{\tan^2 \frac{\alpha}{2} + 1}}; \\ B &= \frac{\pi}{\alpha} \arctan \frac{x + y \tan \frac{\alpha}{2}}{y - x \tan \frac{\alpha}{2}}. \end{aligned}\right\} \qquad (22)$$

The present solution holds for both *symmetrical and asymmetrical* shapes of the dam cross-section.

For various positions of the slopes, the problem is solved by clockwise rotation of the coordinate axes through an angle $\gamma$:

$$\gamma = \arctan \frac{1}{n_2} - \frac{\pi - \alpha}{2}, \tag{23}$$

where $n_2$ is the position of the downstream slope (Fig. 17).

The problem is solved on the assumption that the dam is constructed on an infinitely deep layer of frozen soil.

For engineering calculations, a graph is used to determine the width of the thawed zone in the dam foundation (Fig. 18):

$$l = BK_0, \tag{24}$$

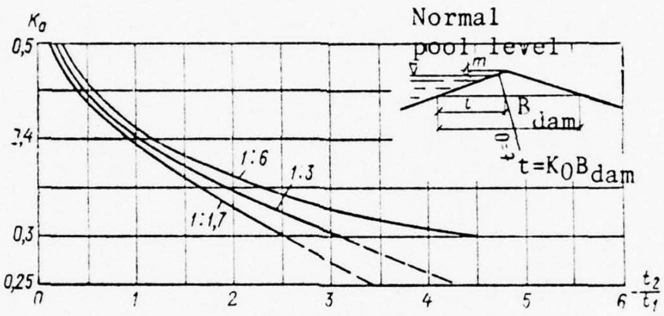


Fig. 18. Values of $K_0$ for use in finding the zero-degree (Celsius) isotherm, as a function of the ratio of mean annual water and air temperatures, for frozen earth dams with slopes of 1 : 1.7 ($\alpha$ = 120°), 1 : 3 ($\alpha$ = 140°) and 1 : 6 ($\alpha$ = 160°). Here $m/H_{dam}$ = 1/4, where $m$ is the freeboard above normal pool elevation.

where $l$ is the width of the thawed zone in the dam foundation; $B$ is the width of the dam at the foundation; and $K_0$ is the criterion of limiting thawing of the dam under steady-state conditions.

Values of $K_0$ have been determined for slopes of 1 : 1.7, 1 : 3 and 1 : 6, and graphs have been drawn that allow $K_0$ to be found for differing positions of the dam slopes, both symmetrical and asymmetrical. The shape of the dam is characterized by the angle $\alpha$; the crest width of the dam is not taken into account (see Fig. 17).

Figures 18 and 19 give curves for finding $K_0$. In Fig. 18 the ratio $m/H_{dam}$ is 1/4, in Fig. 19 $m/H_{dam}$ = 1/6, where $m$ is the freeboard above normal pool elevation and $H_{dam}$ is the height of the dam.

The quantity $K_q$ allows for the effect of heat flux from the interior of the earth on the position of the zero-degree (Celsius) isotherm. $K_q$ depends on the thickness of the frozen layer $h$, the height of the dam $H$, the freeboard above normal pool elevation, and the temperatures of the reservoir water $t_1$ and the outdoor air $t_2$.

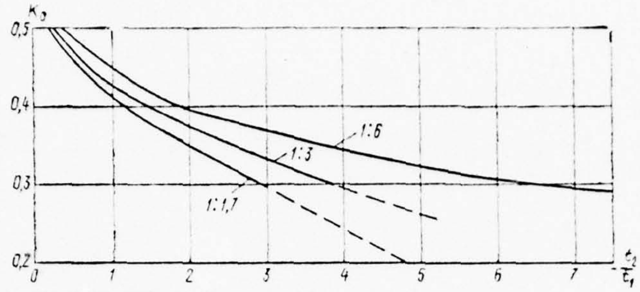


Fig. 19. Values of $K_0$ for use in finding the zero-degree (Celsius) isotherm, as a function of the mean annual water and air temperatures, for frozen earth dams with slopes of 1 : 1.7, 1 : 3 and 1 : 6. Here $m/H_{dam}$ = 1/6, where $m$ is the freeboard above normal pool elevation.

The width of the thawed zone, with allowance for the upward heat flux in the dam foundation, will be determined by the relation

$$l_q = K_0 K_q B. \tag{25}$$

Fig. 20 gives curves for determining $K_q$. Studies of the non-steady-state temperature regime in a number of frozen dams have shown that, for dam heights up to 20 m and various combinations of outdoor temperatures and frozen-soil conditions, the thermal effect of the reservoir on the temperature field of the natural massif under the dam is perceptible to depths of 25-35 m. For this reason, the natural temperature of the frozen soil at a depth of 35-40 m can be taken as the lower boundary condition. When the curve of Fig. 20 is used to find $K_q$ for dams 20-30 m high, the calculated thickness of the frozen layer in the foundation can be taken as not greater than 30-40 m.

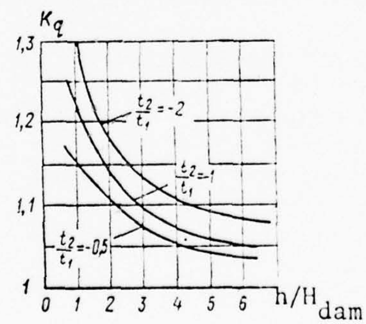


Fig. 20. Curves for determining $K_q$ (heat flux from the interior of the earth for a homogeneous dam).

5. Thermotechnical calculation
of the frozen cutoff in a homogeneous earth dam
without allowance for interaction of the columns

The calculation of the dynamics of growth of the frozen-soil cylinder around an air-cooled freezing column is based on the following assumptions:

The dam is built from a homogeneous soil. The initial above-freezing temperature of the soil and its moisture content, density and thermophysical properties are averaged over the height, length and width of the dam.

The height-averaged temperature of the outer surface of the freezing column is constant throughout the period of winter cooling.

Heat exchange between the column and the soil is considered only in a horizontal plane; the heat fluxes are directed radially to the column and the frozen cylinder concentric with it.

The freezing temperature of the soil is assumed equal to 0°C. No unfrozen water remains in the soil.

Seepage, and the convective heat transfer that would accompany it, do not occur within the frozen cutoff.

All the heat released on cooling of the growing frozen cylinder and the surrounding thawed soil massif is concentrated on the freezing boundary, that is, on the outer surface of the frozen cylinder.

The non-steady-state temperature regime is considered as a sequence of steady states.

The joint operation and thermal interaction of the columns in the frozen cutoff are not taken into account (these factors would shorten the freezing time).

*(a) Freezing for one winter*

The following equation, usable for practical purposes, gives the radius of the frozen cylinder:

$$R_1 = \sqrt[3]{\frac{3\tau_1 t_c r_c \lambda_{fr}}{q}} + 0.5r_c, \qquad (26)$$

where $R_1$ is the radius of the frozen cylinder after the first winter of cooling, m; $\tau_1$ is the length of the first winter cooling period, hr; $t_c$ is the height-averaged temperature of the outer surface of the column over the cooling period, °C; $r_c$ is the outer radius of the column, m; $\lambda_{fr}$ is the thermal conductivity of frozen soil, kcal/m-hr-degree; and $q$

is the quantity of heat released upon the freezing of one cubic meter of soil from $t_i$ [see text after Eq. (28)] to $t_{lim}$ and carried off by air moving in the column. The relation

$$q = \rho W_v = 80{,}000\ W_t \gamma_{sk} \tag{27}$$

gives $q$, where $W_v$ is the volumetric moisture content ($W_v = W_t \gamma_{sk}$); $W_t$ is the total moisture content by weight, a fraction; $\gamma_{sk}$ is the specific gravity of the soil skeleton, ton/m$^3$; and $\rho$ is the latent heat of fusion of ice, equal to 80 kcal/kg.

*(b) Freezing for two or more winters*

For prolonged freezing (two or more winters), the relation among cooling time, radius of the frozen cylinder, and the other data can be defined as

$$\left.\begin{aligned} \tau &= \frac{80\,000 W_v + t_i\ C_{th} - 0{,}33 t_c C_{fr}}{4 t_c \lambda_{fr}} A; \\ A &= \left( 2R_2^2 \ln \frac{R_2}{r_c} - 2R_1^2 \ln \frac{R_1}{r_c} - R_2^2 + R_1^2 \right), \end{aligned}\right\} \tag{28}$$

where $W_v$ is the moisture content by volume, a fraction; $t_i$ is the initial soil temperature, most often above but near freezing ($0°C \leq t_i \leq 1°C$); $C_{th}$ and $C_{fr}$ are the volumetric specific heats of thawed and frozen soils, kcal/m$^3$-degree; $R_1$ is the radius of the frozen cylinder at the end of the preceding freezing period, determined by Eq. (26); and $R_2$ is the radius of the frozen cylinder at the end of the second cooling period, m. For the other symbols see the text after Eq. (26).

Eq. (28) can be used for determining the radius of the frozen cylinder when the cooling system operates for several cooling periods; in that case $\tau = \tau_1 + \tau_2 + \ldots + \tau_n$.

The calculated mean temperature of the outer column, $t_c$, should be determined by the formula

$$t_c = t_{oa} + 4°C, \tag{29}$$

where $t_{oa}$ is the mean temperature of outdoor air over the cooling period (November to March).

*(c) Thermotechnical calculation of the frozen cutoff with allowance for the interaction of columns (two-dimensional problem)*

To determine the operating time for the freezing system--up to the moment when the frozen cylinders merge to form a continuous frozen cutoff of a given thickness--Eq. (30), which allows for the interaction of adjacent columns, should be used.

The following assumptions are made to convert from the three-dimensional

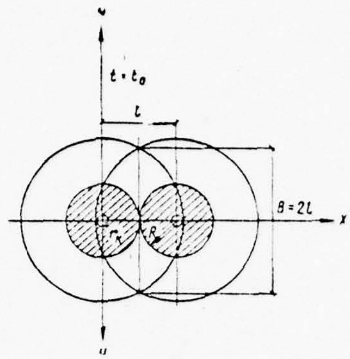


Fig. 21. Computational scheme for determining the sizes of the frozen cylinders.

problem to the two-dimensional problem in a plane perpendicular to the column axis (Fig. 21):

The vertical temperature gradient in the soil equals zero.

The air temperature in a cross-section perpendicular to the column axis is constant over the period being calculated.

The initial temperature and physical characteristics of the soil are uniform throughout the soil massif.

The freezing problem with interacting columns cannot be solved analytically; for this reason, similarity theory and methods were used. Many special cases were solved on the hydrointegrator and then generalized.

This procedure yielded a starting numerical equation that links, in general form, the Fourier number with other expressions:

$$\mathrm{Fo} = f(\mathrm{Bi},\ \mathrm{KO}[t_{air}/t_0],\ l/r_c). \qquad (30)$$

Here $\mathrm{Fo} = \lambda_{fr}\tau/C_{fr}r_c^2$ is the Fourier number (a dimensionless time); $\mathrm{Bi} = \alpha_1 d_c/\lambda_{fr}$ is the Biot number; and $\mathrm{Ko} = q/C_{fr}[t_{air}]$ is the Kossovich number.

Also in Eq. (30), $\lambda_{fr}$ is the thermal conductivity of the frozen soil, kcal/m-hr-degree; $C_{fr}$ is the volumetric specific heat of the frozen soil, kcal/m$^3$-degree; $r_c$ and $d_c$ are the outside radius and diameter of the freezing column, m; $\tau$ is the operating time for the freezing system, hr; $l$ is the spacing of columns, m; $t_{air}$ is the air temperature in the column, degrees, averaged over the calculated period; $t_0$ is the initial soil temperature, degrees; $q$ is the amount of heat used in phase transformations of soil moisture per cubic meter of soil, kcal/m$^3$; and $\alpha_1$ is determined by the formula given in Appendix 3, Example 5.

If only the time to merging of the ice-soil cylinders for given $l$ (or the time to formation of a frozen cutoff $2l$ wide) is needed, the nomogram

for the relation Fo = $f(l/r_c)$ can be used (Fig. 22).

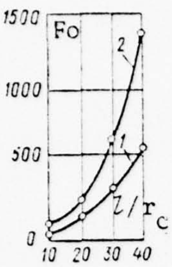


Fig. 22. Fourier number Fo versus the dimensionless column spacing. (1) Moment when frozen cylinders merge; (2) formation of an ice-soil wall $2l$ wide.

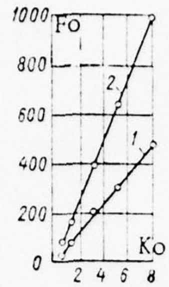


Fig. 23. Operating time for the freezing system--to merging of frozen cylinders--versus the Kossovich number. (1) $l/r_c$ = 15; (2) $l/r_c$ = 20.

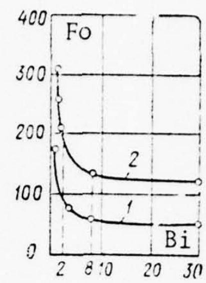


Fig. 24. Operating time for the freezing system--to merging of the frozen cylinders--versus the Biot number. (1) $l/r_c$ = 15; (2) $l/r_c$ = 20.

Experimental and theoretical studies have shown that the operating time for the freezing system--up to the merging of the ice-soil cylinders and the formation of a continuous ice-soil wall--is directly proportional to Ko (Fig. 23). This quantity depends on the amount of heat used in the phase transformation of soil moisture, the temperature of the heat-transfer agent, and the specific heat of the soil.

Solution of the series of problems on the hydrointegrator yielded functions approximating the relations between the dimensionless time (Fo) and each of the criteria determining the soil-freezing process for two neighboring columns. Formulas were obtained by which Fo can be determined for the moment when the frozen cylinders merge and the moment when a wall $2l$ wide forms.

Because these relations are complicated, nomograms have been produced to replace them as means of determining Fo = $f$(Bi), Fo = $f$(Ko) and Fo = $f(l/r_c)$.

When optimum dimensions for the freezing system are to be chosen, the value of Bi should be checked. At small Bi the operating time for the freezing system--up to the merging of the ice-soil cylinders--increases sharply. Increasing Bi beyond six to eight has no practical effect (Fig. 24).

$$2 < \text{Bi} < 8. \tag{31}$$

The heat-transfer coefficient $\alpha_1$ has a substantial effect on Bi.

The value of $\alpha_1$ depends on the velocity of the heat-transfer agent (air).

Fig. 25 gives a nomogram for the most probable range of variation of the expressions Fo, Bi and Ko.

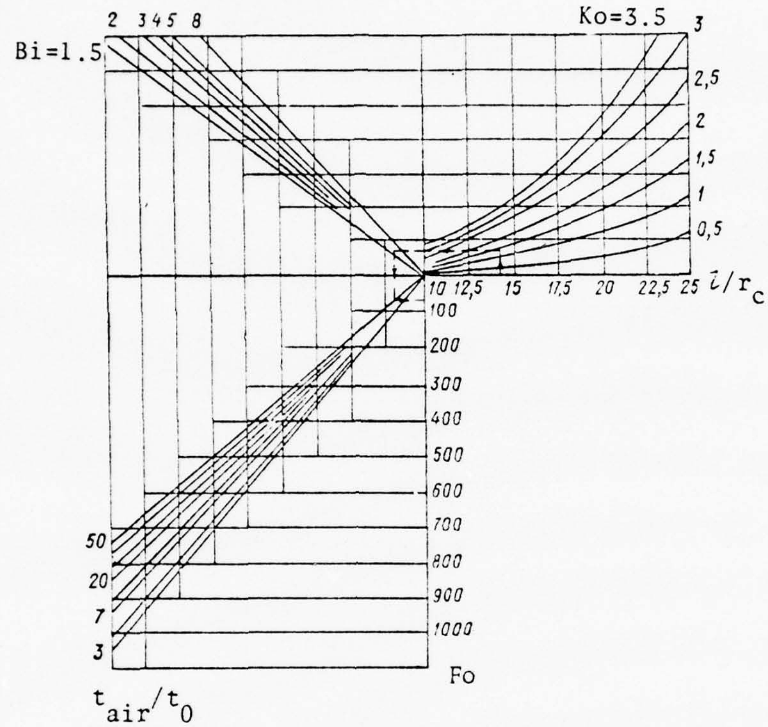


Fig. 25. Nomogram of most probable variations in the determining criteria.

In the determination of Fo, Bi and Ko, it is assumed that the calculation for a single freezing column has already been performed.

The optimality of the parameters taken for the air-cooled freezing column can be verified by the test of S. M. Filippovskiy:

$$\lambda_{fr} h / C_{air} Q \rho_t < 0.3, \qquad (32)$$

where $C_{air}$ is the volumetric specific heat of air; $Q$ is the air flow; $h$ is the column length; and $\rho_t$ is the thermal resistance of the column walls.

Appendix 3, Example 5, shows a check on optimality of the parameters of a freezing column. Appendix 3, Example 6, gives a sample calculation of the time to merging of the frozen cylinders and to formation of a frozen wall, with allowance for the interaction of columns.

*6. Determination of air velocity and blower capacity*

For the determination of air velocity in the freezing columns it is necessary to know:

(a) The radius of the frozen cylinder (after the first winter cooling period $\tau_1$), m;

(b) The outside radius of the column $r_c$, m.

(c) The mean length of the columns $h_c$, m, and the number of columns $n$.

The volume of the frozen cylinder around each column will be

$$V_{cyl} = \pi(R_1^2 - r_c^2)\ h_c. \tag{33}$$

The volume of the frozen wall formed in time $\tau_1$ by the operation of a system of $n$ columns equals

$$V = V_{cyl}n. \tag{34}$$

The quantity of heat liberated by soil freezing in the whole volume of the frozen wall, with all columns operating, is

$$q_3 = qV = 80{,}000W_0; \tag{35}$$

$$W_0 = W_t\gamma_{sk},$$

where $W_t$ is the total moisture content of the soil, a fraction; and $\gamma_{sk}$ is the specific gravity of the soil skeleton, ton/m$^3$.

For a system of $n$ columns, the total transmission of cold into the soil as cold outdoor air moves through the annular space in the column is

$$Q^* = \Phi\gamma_{air}C'_{air}\ (t_{in} - t_{out}), \tag{36}$$

where $\Phi$ is the total volume of air passed through all columns of the frozen cutoff over the freezing period $\tau_1$; $\gamma_{air}$ is the specific gravity of air, kg/m$^3$ (at $t$ = 0); $C'_{air}$ is the specific heat of air, kcal/kg-degree; and $t_{in}$ and $t_{out}$ are the inlet and outlet air temperatures.

When cold air is injected through the inner pipe and exhausted through the annular clearance between the inner and outer pipes of the column, the relation between the inlet and outlet temperatures and the air temperature at the column bottom is

$$t_{out} - t_{bot} = (t_{out} - t_{in})\ (1 - \pi K_1 h_c/\Phi_1 C_{air}), \tag{37}$$

where $t_{out}$, $t_{in}$ and $t_{bot}$ are the air temperatures at the outlet, inlet and bottom of the freezing column, °C; $\Phi_1$ is the air flow, $m^3$/hr; $C_{air}$ is the volumetric specific heat of air (0.325 kcal/$m^3$-degree); $K_1$ is the linear heat-transfer coefficient, kcal/$m^3$-hr-degree; and $h_c$ is the column length, m.

For steel, plywood and rigid-PVC inner pipes, $K_1$ has the values 1.38, 1.03 and 0.966 respectively.

If $Q^*$ and $q_3$ are assumed equal, then Eqs. (35) and (36) yield the desired value of $\Phi$.

To find the velocity of the air, it is necessary to set the capacity of all the blowers; the total capacity of all blowers in the system will be

$$P = \Phi/\tau_1, \tag{38}$$

where $\tau_1$ is the length of winter ventilation of the columns, hr.

If the system of freezing columns is divided into $m$ sections, the capacity of each blower serving one of the $m$ sections is

$$P' = P/m. \tag{39}$$

If the annular cross-section of a column $\omega$ is known (in $m^2$) and the number of columns in the section $N$ is known, then the mean velocity of air in the annular space is given by

$$V = \frac{P'}{3600N\omega}, \tag{40}$$

where $N = n/m$ and $\omega = \pi[(d_2/2)^2 - (d_1/2)^2]$, m; and $d_2$ and $d_1$ are the inside and outside diameters of the inner pipe of the column, both in meters.

On the basis of field data, $V_{air}$ should not be less than 3 m/sec. The optimum value is $V_{air}$ = 5-9 m/sec.

If $V_{air}$ is given in the range $2 \leq V_{air} \leq$ 8-9 m/sec, then Eq. (40) can be used to determine the necessary blower capacity for each section.

Given the blower capacity $P'$, the formula

$$V_{air} = \frac{0.35P}{d_2^2 - d_1^2} \tag{41}$$

gives a rough value for the air velocity in the annular space.

Appendix 3, Examples 7 and 8 show determinations of the air velocity and blower capacity.

7. Mean temperature of the frozen cutoff

Calculations of the stability and strength of the frozen core and of the dam cross-section as a whole require a determination of the height-average temperature of the frozen cutoff.

The methods reviewed above for the calculation of frozen cutoffs do not include recommendations on the determination of mean temperatures of the frozen cutoff.

For practical purposes, the height-average temperature of the cutoff can be approximately determined from the relations for:

(a) Mean temperature, over the cutoff volume, when the frozen cylinders completely merge,

$$t_m = t_c(0.32 + 0.8d/l - 0.2l/B). \qquad (42)$$

(b) Mean, over the cutoff, of temperature at the intersection of the cutoff longitudinal axis with the plane of merging of the cylinders,

$$t_{mer} = t_c\ (0.73 - 0.55l/B + d/l). \qquad (43)$$

In Eqs. (42) and (43), $d$ is the outside diameter of the column, m; $l$ is the axis-to-axis spacing between columns; $B$ is the minimum thickness of the cutoff in the plane in which the cylinders merge, m; and $t_c$ is the design temperature of the outer surface of a column, averaged over the period of ventilation.

Temperatures for the Irelyakhsk dam were calculated by Eqs. (42) and (43) and the results were compared with field data (Appendix 3, Example 4); the agreement was satisfactory.

8. Convective heat exchange
between the soil surface and the air

If a temperature gradient exists between two parts of the dam embankment, heat transfer from one section to the other can take place (a) by conduction through the sections and their contacts and (b) by molecular heat conduction (convection) in the medium filling the gap between particles (water or air). Thus heat exchange in the embankment reduces to conduction and convection processes.

In approximate calculations of the temperature regimes in the dam embankment and foundation, the starting point is usually just the heat-conduction equation, since convection is assumed not to have much effect.

However, as recent experimental and theoretical studies have shown, convection should be taken into account in the embankments of earth-rock dams.

The variation of temperature, boundary conditions remaining constant, is determined by three thermophysical characteristics: the thermal conductivity $\lambda$, the specific heat $C$, and the thermal diffusivity $a$.

In dams, where coarse granular materials, shingle and rockfill may be used, heat transfer is due not just to conduction but, under certain conditions, to convection as well.

Natural convection may have great importance for non-filter dams. Convective heat exchange can be accounted for, in an approximate way, by the introduction of "effective thermophysical characteristics" of the material. For air to start moving in the embankment, it is necessary that the lifting force, namely the critical temperature difference (due to inhomogeneities in the temperature field), be somewhat greater than the resistance of the embankment material to the motion of air.

In the downstream shell of a rockfill dam, a variable-temperature air stream forms and continues until the temperature of the downstream shell approaches the outdoor air temperature.

The effective thermal conductivity is defined by

$$\lambda_{eff} = \phi\lambda_t, \tag{44}$$

where $\phi$ is the coefficient of augmentation of the thermal conductivity by convective fluxes.

Rough values of $\phi$ for heat transfer by air and water can be obtained from the curves of Fig. 26* as a function of the permeability coefficients of the embankment soils and the height $h$ of the dam for the cross-section under consideration.

The following example clarifies the effect of convective heat exchange, which increases the thermal conductivity for coarse-fragmented materials.

The permeability coefficient of a rockfill is $K$ = 20 m/day; $h$ = 30 m (height of the dam at a distance of 30 m from the crest); and

$$\lambda_t = 2.8 \text{ kcal/hr-m-degree} = \frac{2.8 \cdot 24}{1000} = 0.0672 \text{ Mcal/day-m-degree}.$$

For these $K$ and $h$ values, Fig. 26 gives

$$\lambda_t\ (\phi - 1) = 0.02 \text{ Mcal/day-m-degree};$$

$$\phi - 1 = \frac{0.02}{0.0672} = 0.296;\ \phi = 1 + 0.296 = 1.296.$$

*Translator's note: Original had "Fig. 2."

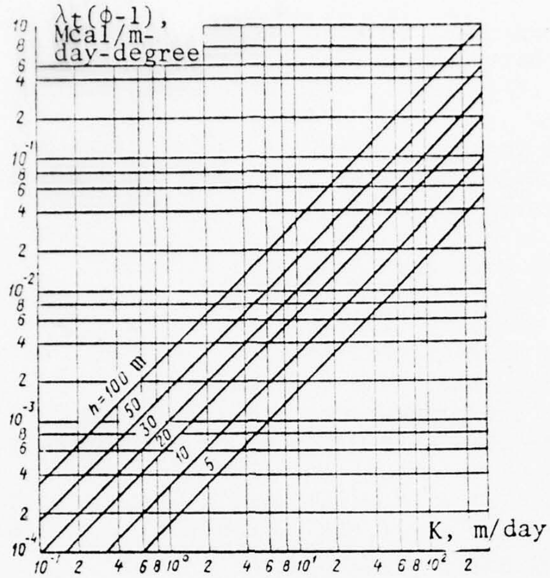


Fig. 26. $\lambda_t$ ($\phi$ - 1) as a function of $K$ and $h$.

$$\lambda_{eff} = \phi\lambda_t = 1.296 \cdot 0.0672 = 0.0874 \text{ Mcal/day-m-degree.}$$

The value of $\phi$ in the rockfill should be taken into account.

Let us consider the effect of convective heat exchange on the thermal conductivity for sandy loams.

The soils have a permeability coefficient of $K$ = 0.3 m/day; $\lambda_t$ = 2.3 kcal/m-hr-degree; $\lambda_t$ = 0.055 Mcal/day-m-degree; and $h$ = 30 m.

For these $K$ and $h$ values, Fig. 26 yields

$$\lambda(\phi - 1) = 3 \cdot 10^{-4};$$

$$\phi - 1 = \frac{0.0003}{0.055} = 0.00545;$$

$$\phi = 1 + 0.005 = 1.005.$$

In this case, $\phi$ need not be taken into account.

The value of $\phi$ should be taken into account if $\phi \geq 1.1$.

Appendix 3.
Examples of approximate calculations:
Temperature regimes of dam embankment and foundation and reservoir bottom.

*Example 1. Dynamics of thawing of permafrost under the reservoir bottom (position of zero-degree [Celsius] isotherm)*

Given:

$t_1$, the water temperature at the reservoir bottom, +6°C.

$t_2 = t_{soil}$, soil temperature, -4°C.

$C_{perm}$, volumetric specific heat of the permafrost, 400 kcal/m$^3$-degree.

$\lambda_t$, thermal conductivity of thawed soil completely saturated with water, 1.25 kcal/m-hr-degree.

$W_t$, total moisture (ice) content of soil, 0.2.

$\rho$, latent heat of the phase transformation of soil moisture, 80,000 kcal/ton.

$\tau$, time from filling of the reservoir to the moment under consideration (in hours).

The soil thaws to a depth $x$ in time $\tau$:

$$x = \sqrt{\frac{2\lambda_t t_1 \tau}{0.9\rho W_t + C_{perm} t_{soil}}} = \sqrt{\frac{2 \cdot 1.25 \cdot 6\tau}{0.9 \cdot 80{,}000 \cdot 0.2 + 400 \cdot 4}} = \frac{\sqrt{\tau}}{33};$$

$$\tau = 1 \text{ yr} = 8750 \text{ hr};\ x = \frac{\sqrt{8750}}{33} = 2.85 = 2.9 \text{ m}.$$

Table 12 gives the $x$ values for various $\tau$.

Table 12.

| $\tau$, years | $x$, m |
|---|---|
| 1 | 2.85 |
| 5 | 6.35 |
| 10 | 8.99 |
| 20 | 12.6 |
| 30 | 20 |

*Example 2. Steady temperature state in soil of the reservoir bottom*

See Fig. 27.

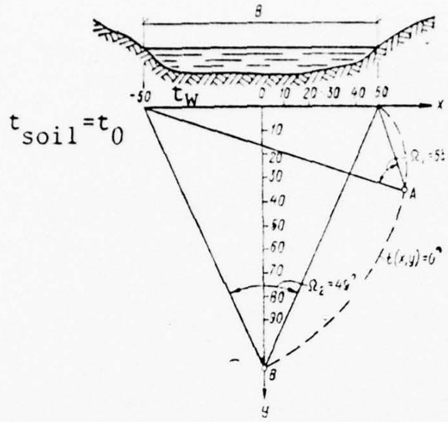

Fig. 27. Angles of opening $\Omega$ for points $A$ and $B$ in the determination of the limiting thawing zone under the reservoir bottom.

Given the thermophysical characteristics of the reservoir bottom:

$\lambda_{fr}$ = 1.8 kcal/hr-m-degree.

$\lambda_{th}$ = 1.5 kcal/hr-m-degree.

$t_i$ = +4°C.

$t_{soil} = t_0$ = -2°C.

$B$ = 100 m.

To find the soil temperature at points $A$ and $B$ with coordinates

$A$ ($x$ = 60 m, $y$ = 35 m), $B$ ($x$ = 0, $y$ = 110 m).

By Eq. (17),

$$t_{(x,y)} = \frac{\frac{\lambda_{th}}{\lambda_{fr}} t_i - t_0}{\pi} \Omega + t_0,$$

where $\Omega$ is the angle of opening for points $A$ and $B$, determined graphically (see Fig. 24); $\alpha_A$ = 55°, or in radians $\alpha_A$ = 55·0.0174 = 0.955; and $\alpha_B$ = 49°, or in radians $\alpha_B$ = 49·0.0174 = 0.85. Then

$$t_A = \frac{\frac{1,5}{1,8}\,4-(-2)}{3,14}\,0,955 - 2,0 = -0,38^\circ\ \mathrm{C};$$

$$t_B = \frac{\frac{1,5}{1,8}\,4-(-2)}{3,14}\,0,85-2 = -0,68^\circ\ \mathrm{C}.$$

In the center of the reservoir the maximum thawing is determined by Eq. (18):

$$x = 0,5B\cot\left[\frac{\pi}{2}\cdot\frac{\lambda_{th}t_0}{\lambda_{th}t_0 - \lambda_{fr}t_1}\right] = 0,5\cdot 100\cot\left[\frac{3,14}{2}\times\right.$$

$$\left.\times\frac{1,5(-2)}{1,5(-2)-1,8(+4)}\right] = 102\ m.$$

*Example 3. Position of zero-degree (Celsius) isotherm in frozen bank slope after filling of the reservoir, as a function of time*

See Fig. 28.

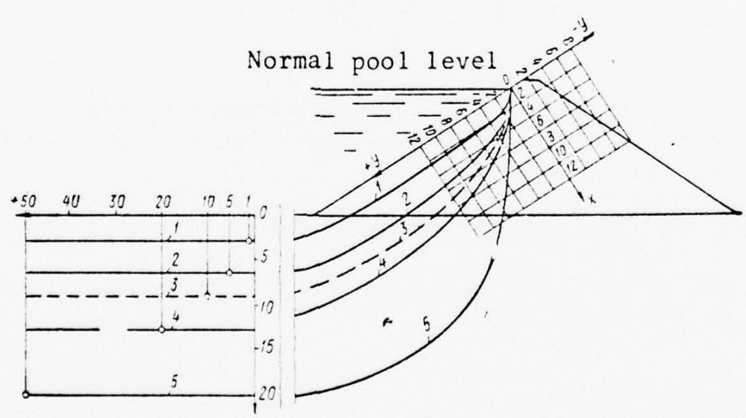


Fig. 28. Position of the boundaries between thawed and frozen soil within the slope portion of the soil adjacent to the reservoir and under the reservoir bottom, for $\lambda_{fr}t_2 > \lambda_{th}t_1$. (1) $\tau$ = 1 yr, $x$ = 2.9 m; (2) $\tau$ = 5 yr, $x$ = 6.4 m; (3) $\tau$ = 10 yr, $x$ = 9 m; (4) $\tau$ = 20 yr, $x$ = 12.6 m; (5) $\tau$ = 50 yr, $x$ = 20 m.

Given:

$t_2$, the soil temperature in the bank slope above the water level, -5°C.

$t_1$, the mean annual water temperature in the reservoir, +4°C.

$t_{perm}$, the initial temperature of the permafrost, -4°C.

$\lambda_{th}$ = 1.5 kcal/hr-m-degree.

$\lambda_{fr}$ = 1.8 kcal/hr-m-degree.

$W_v$ = 0.2.

$\rho$ = 80,000 kcal/m$^3$.

To construct zero-degree (Celsius) isotherms for times $\tau$ = 1, 5 and 20 yr.

One of Eqs. (19) and (20) gives the boundaries of the thawed and frozen zones:

$$y = x\sqrt{\frac{\lambda_{fr} t_2}{\lambda_{th} t_1 - \frac{Q}{2\tau} x^2} - 1};$$

$$y = -x\sqrt{\frac{\lambda_{th} t_1}{\lambda_{fr} t_2 + \frac{Q}{2\tau} x^2} - 1}.$$

Since $\lambda_{fr} t_2 > \lambda_{th} t_1$ $(1.85 \cdot 5 > 1.5 \cdot 4)$, the boundary of the frozen zone is found by the first equation alone. The value of $Q$ in this relation is determined by the formula

$$Q = 0.9\rho W_v + C_{perm} t_{soil} = 0.9 \cdot 80{,}000 \cdot 0.2 + 400 \cdot 4 = 16{,}000 \text{ kcal/m}^3.$$

In Eqs. (19) and (20), $t_1$, $t_2$ and $t_{soil}$ are the absolute values of the temperatures in degrees Celsius.

At $\tau$ = 1 yr,

$$y = x\sqrt{\frac{1,8 \cdot 5}{1,5 \cdot 4 - \frac{16000}{2 \cdot 8750} x^2} - 1} = x\sqrt{\frac{9}{6 - 0,912\, x^2} - 1}.$$

The calculations are similar for $\tau$ = 5 and 20.

For $\tau = \infty$,

$$y = x\sqrt{\frac{9}{6} - 1} = x \cdot 0.71.$$

Table 13 gives $y$ as a function of $x$ for various times $\tau$.

Table 13.

| $x$ | Value of y for $\lambda_{frt2} > \lambda_{tht1}$ | | | |
|---|---|---|---|---|
| | 1 yr | 5 yr | 20 yr | $\infty$ |
| 1 | 0.877 | 0.739 | 0.715 | — |
| 1.5 | 1.697 | — | — | — |
| 2 | 3.362 | 1.628 | 1.478 | — |
| 3 | — | 3.102 | 2.346 | 2.13 |
| 3.5 | — | 4.134 | — | — |
| 4 | — | — | 3.372 | — |
| 5 | — | 11.53 | 4.625 | — |
| 6 | — | — | 6.210 | 4.26 |
| 9 | — | — | 15.48 | — |

Fig. 28 shows the orientation of the $x$ and $y$ axes.

*Example 4. Thermotechnical calculation of dynamics of growth of a frozen cutoff in an earth dam*

To determine the radii of frozen cylinders at various times during winter cooling, for the dam on the Irelyakh River (height 10 m). Fig. 29 gives the cross-section of the dam and also shows the position of a freezing column and temperature-measuring wells.

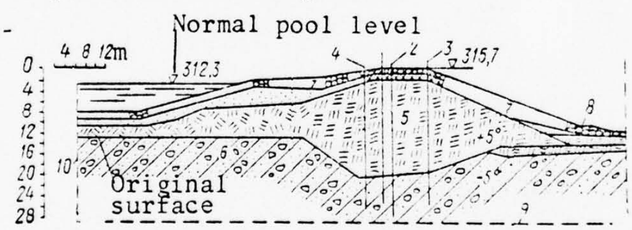


Fig. 29. Cross-section used in calculations of the dam on the Irelyakh River. (1) Freezing column; (2, 3, 4) temperature-measuring wells; (5) clay loam; (6) original semi-rocky soils; (7) slope protection from crushed stone and rockfill; (8) peat-moss layer; (9) lower limit of cross-section for calculation (42 m below crest elevation); (10) vertical boundary of cross-section for calculation.

Given:

$l$, the center-to-center spacing of columns, 1.5 m.

$h$, the height of a column (average), 20 m.

$t_c$, the mean temperature of the outer surface of a column, $-20° + 4° = -16°$. (Calculated outdoor-air temperature in the ventilation period is $-20°$. In preliminary calculations, $t_c$ can be assumed equal to $t_i + 4°C$.)

$r_c$, outside radius of a column, 0.11 m.

$\lambda_{fr}$, thermal conductivity of frozen soil, 1.8 kcal/m-hr-degree.

$q$, quantity of heat liberated when 1 $m^3$ of soil is cooled from $t_i$ to $t_{lim}$.

$t_i$, initial, above-freezing average soil temperature, +1°C (assumed from local conditions).

$t_{lim}$, limiting temperature to which the central part of the dam should be cooled, -1°C.

$W_v$, volumetric moisture content of soil, 0.15.

$\gamma_{sk}$, density of soil skeleton, 1.7 ton/$m^3$.

$t_1$, winter cooling time (first winter from startup of system).

$R_1$, radius of frozen cylinder over the first winter of operation of the system, depending on the column radius and other factors and determined by Eq. (26):

$$R_1 = \sqrt[3]{\frac{3\tau_1 t_c r_c \lambda_{fr}}{q}} + 0{,}5\, r_c,$$

where $q$ = 80,000 $W_t \gamma_{sk}$, in kcal/$m^3$.

Since two unknowns $R_1$ and $\tau_1$ appear in Eq. (26), one of them should become a given.

Let us give $\tau_1$ = 1000 hr ≈ 42 days = 1 month 12 days. Then the radius of the frozen cylinder will equal

$$R_1 = \sqrt[3]{\frac{3\cdot 1000\,(--16)\,0{,}11\cdot 1{,}8}{20400}} + 0{,}5\cdot 0{,}11 = 0{,}78 + 0{,}06 = 0{,}84 \text{ m},$$

where $q$ = 80,000·0.15·1.7 = 20,400 kcal/m$^3$.

This time is sufficient, with some reserve, for the merging of frozen cylinders with a radius of $R_1$ = 0.75 m and $l$ = 1.5 m.

Eq. (28) gives the time $\tau_2$ for the subsequent freezing period:

$$\tau_2 = \frac{80\,000\,W_0 + t_i\ \ C_{th} - 0{,}33\,t_c\,C_{fr}}{-\,4t_c\,\lambda_{fr}} \times \left(2R_2^2 \ln \frac{R_2}{r_c} - 2R_1^2 \ln \frac{R_1}{r_c} - R_2^2 + R_1^2\right). \tag{45}$$

We take $R_1$ = 1 m for the second freezing period during the second and following winters.

Let us have given $R_2$ = 6 m; $R_2^2$ = 36 m$^2$; $t_c$ = -20°C (assumed for the second winter and subsequent cooling periods); and $t_i$ = +1°C.

Here $C_{th}$ and $C_{fr}$ are the averaged volumetric specific heats of thawed and frozen soil; $C_{th}$ = 700 kcal/m$^3$-degree and $C_{fr}$ = 456 kcal/m$^3$-degree. Also, $\tau_2$ is the length of the second (or any subsequent) winter cooling period.

$$\tau_2 = \frac{80\,000 \cdot 0{,}15 + (+1)\,700 - 0{,}33\,(-20)\,465}{-4\,(-20)\,1{,}8} \left(2 \cdot 36 \ln \frac{6}{0{,}11} - \right.$$
$$\left. -2 \cdot 1^2 \cdot \ln \frac{1{,}0}{0{,}11} - 36 + 1\right) = \frac{9640}{140} \cdot 244{,}6 = 16\,400 \text{ hr};$$
$$\tau_2 = \frac{16\,400}{24 \cdot 30} = 22{,}8 \text{ months}.$$

Then the total cooling time is

$$\tau = \tau_1 + \tau_2 = 1.3 + 22.8 \simeq 24 \text{ months}.$$

If the initial soil temperature is assumed to be $t_i$ = +5°C and the limiting temperature $t_{lim}$ is taken as -1°C, then the calculated cooling period for the dam embankment and foundation is $\tau$ = 29-30 months.

Field data (Fig. 30) show that nearly the same length of time was needed for the cooling of a dam embankment and foundation to $t_{lim}$ = -1°C.

Figure 30 presents the temperature variations for the embankment and foundation, as found (a) by theoretical calculations on a computer for $t_{lim}$ = -0.1°C and (b) from field data.

Figure 31 shows the temperature distribution in a cross-section of the dam on the Irelyakh River, after freezing for three winters, as recorded in field data. Figure 32 gives the temperature distribution in the same cross-section as calculated on the computer.

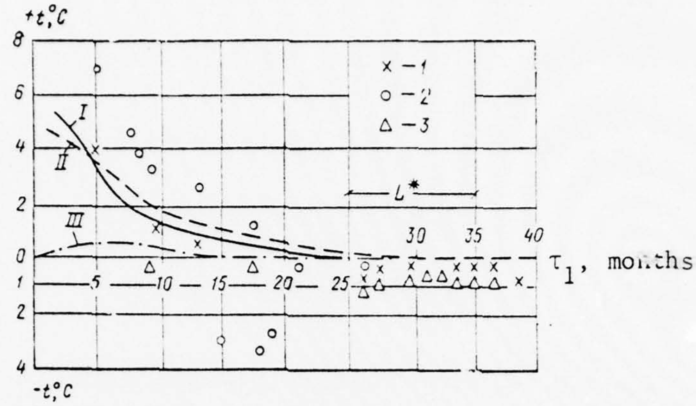


Fig. 30. Calculated temperature curves for a dam embankment and foundation, and temperature values at coincident points measured in the field (well No. 5).
(I) Calculated soil temperature 7 m below the crest; (II) calculated soil temperature 11 m below the crest; (III) calculated soil temperature 19 m below the crest.
(1, 2, 3) Observed values of soil temperature at the same depths. $L^*$ is the period of active freezing (freezing time $\tau$ = 25-30 months).

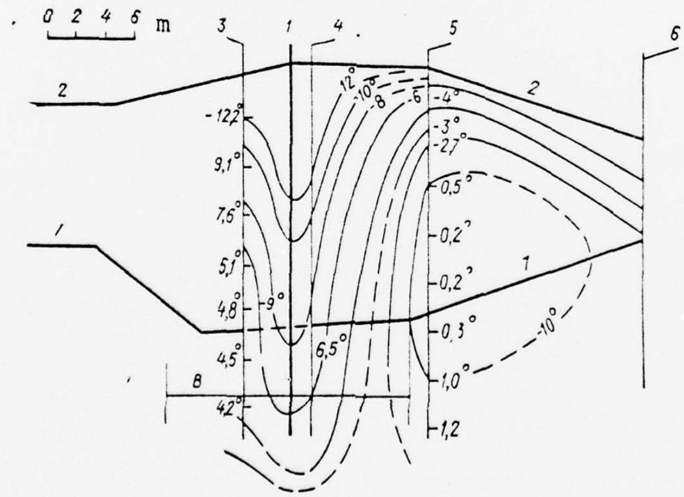


Fig. 31. Temperature field in the central part and foundation of the dam, from field data. $B$ = 16-18 m.
(1) Freezing column; (2) upper outline of dam; (3, 4, 5, 6) temperature-measuring wells; (7) original ground surface.

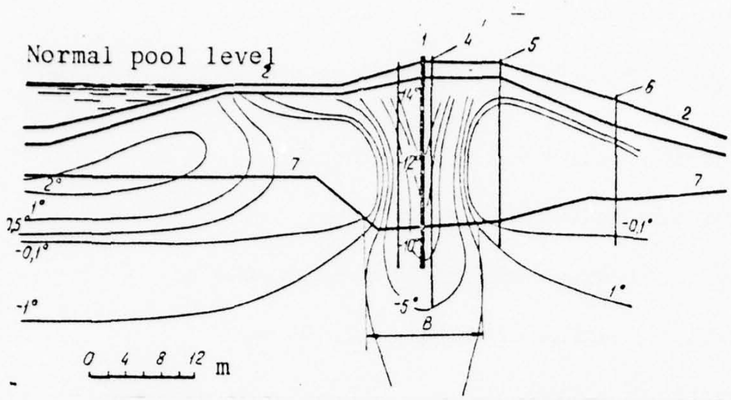


Fig. 32. Temperature field in the central part and foundation of the dam and part of the reservoir bed, March 30, 1967, constructed from computer calculations. $B$ = 10-12 m. (1) Freezing column; (2) upper outline of dam; (3, 4, 5, 6) temperature-measuring wells; (7) original ground surface.

The cross-section in Figs. 31 and 32 lies 25-30 m from the river bed on the left-bank side.

*Comparison of field data and calculations on the formation of a frozen cutoff in the central part of the dam on the Irelyakh River*

(a) A theoretical calculation performed on a computer, for the initial freezing period of the dam, $t_{lim}$ = -0.1°C and $t_i$ = +5°C, gives $B$ = 10 m for the width of the frozen cutoff in the central part and $B$ = 12 m in the dam foundation (see Fig. 32).

The total winter-freezing time is $\tau$ = 18 months.

(b) Field observations give a time of $\tau$ = 10-12 months for the initial freezing of the central part of the dam to $t_{lim}$ = 0.1° and $t_i$ = +5°C.

More stable freezing of the dam embankment and foundation to $t_{lim}$ = -1° (see Fig. 30) required $\tau$ = 25-35 months of winter freezing. The width of the frozen cutoff was 15-16 m (Fig. 31).

(c) A theoretical calculation was performed with Eqs. (26), (27) and (28).

Freezing of the dam embankment and foundation, taking $t_{lim}$ = -1°C and $t_i$ = +5°C, requires $\tau$ = 29-30 months for freezing; the width of the frozen cutoff will be $B$ = 12-14 m.

As the comparison of computed figures and field data shows, the results are all close enough together for practical purposes.

*Example 5. Test for optimality of parameters of the freezing column*

The case taken is a design variant of the dam on the Sytykan River.

Given:

Parameters of the system of columns in the stream-bed part of the dam

$r_c$, outside radius of column, 0.106 m.

$r_1$, outside radius of supply pipe, 0.0665 m.

$\delta_2$, wall thickness of supply pipe, 0.004 m.

$\delta_1$, wall thickness of column, 0.006 m.

$h$, length of column, 31 m.

$\lambda$, thermal conductivity of pipe walls, 40 kcal/hr-m-degree.

$l$, spacing of columns, 1.5 m.

$r_{eq} = r_c - r_1 = 0.106 - 0.0665 = 0.0395$ m.

Parameters of soil in the central part (core) of the dam embankment from clay loam

$W$, moisture content, 20%.

$\gamma$, density, 2 ton/m$^3$.

$\lambda_{fr}$, thermal conductivity of frozen soil, 2 kcal/m-hr-degree.

$C_{fr}$, volumetric specific heat of frozen soil, 480 kcal/m$^3$-degree.

$t_0$, initial temperature of soil, 4°C.

$t_{soil}$, mean calculated temperature of cooling of soil, -8°C.

Parameters of air

$Q$, air flow, 558 m$^3$/hr.

$V_1$, velocity in the annular space in the column, 8.9 m/sec.

$V_2$, velocity in the supply pipe, 12.6 m/sec.

$t_{air}$, air temperature, -31°C.

$C_{air}$, specific heat, 0.3393 kcal/m$^3$-degree.

$\lambda_{air}$, thermal conductivity, 0.01915 kcal/m-hr-degree.

$\nu$, kinematic viscosity, $0.9494 \cdot 10^{-5}$ m$^2$/sec.

Pr, Prandtl number, 0.722.

(The four last characteristics are determined at $t = t_{air}$.)

The formula

$$\lambda_{fr} h / C_{air} Q \rho_{th} < 0.3$$

is the condition for optimality of the parameters of the freezing column. Here

$$\rho_{th} = \frac{\lambda_{fr}}{\lambda} \ln \frac{r_1}{r_1 - \delta_2} + \frac{\lambda_{fr}}{\alpha_2 (r_1 - \delta_2)} + \frac{\lambda_{fr}}{\alpha_1 r_1};$$

$$\alpha_1 = 0{,}015 \frac{\lambda_{air}}{2r_{eq}} \cdot Pr^{0,4} \cdot Re^{0,8} \left( \frac{r_c}{r_1} \right)^{0.25};$$

$$r_{eq} = r_c - r_1;$$

$$R_e = \frac{2V_1 r_{eq}}{\nu_{air}};$$

$$\alpha_2 = 0{,}023 \frac{\lambda_{air}}{2r_1} Pr^{0,4} Re^{0,8},$$

where $S_t$ is the thermal resistance of the column wall.

$$Re = \frac{2V_1 r_{eq}}{\nu_{air}} = \frac{2 \cdot 8{,}9 \cdot 0{,}0395}{0{,}9494 \cdot 10^{-5}} = \frac{0{,}7031}{0{,}000009} = 78122{,}2;$$

$$\alpha_2 = 0{,}023 \frac{\lambda_{air}}{2r_1} Pr^{0,4} Re^{0,8} = 0{,}023 \frac{0{,}01915}{2 \cdot 0{,}0665} \cdot 0{,}722^{0,4} \cdot 78122^{0,8} = $$
$$= 0{,}023 \cdot 0{,}144 \cdot 0{,}877 \cdot 8210{,}0 = 23{,}85;$$

$$\alpha_1 = 0{,}015 \frac{\lambda_{air}}{2r_{eq}} Pr^{0,4} Re^{0,8} \left( \frac{r_c}{r_1} \right)^{0,25} = 0{,}015 \frac{0{,}01915}{2 \cdot 0{,}0395} \times$$
$$\times 0{,}722^{0,4} \cdot 78122{,}2^{0,8} \left( \frac{0{,}106}{0{,}0665} \right)^{0,25} = 0{,}015 \cdot 0{,}2424 \cdot 0{,}877 \times$$
$$\times 8210{,}0 \cdot 1{,}124 = 29{,}42;$$

$$\rho_{th} = \frac{\lambda_{fr}}{\lambda} \ln \frac{r_1}{r_1 - \rho_2} + \frac{\lambda_{fr}}{\alpha_2 (r_1 - \delta_2)} + \frac{\lambda_{fr}}{\alpha_1 r_1} = \frac{2{,}0}{40{,}0} \ln \frac{0{,}0665}{0{,}0665 - 0{,}004} +$$
$$+ \frac{2{,}0}{23{,}85 (0{,}0665 - 0{,}004)} + \frac{2{,}0}{29{,}42 \cdot 0{,}0665} = 0{,}05 \times$$
$$\times 0{,}0583 + 1{,}34 + 1{,}02 = 2{,}36;$$

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$\lambda_{fr}h_1/C_{air}Q\rho_t < 0.3;$

$$\frac{2{,}0\cdot 31{,}0}{0{,}94\cdot 558\cdot 2{,}36}=0{,}138<0{,}3.$$

Here $\rho_t$ is the thermal resistance of the column wall.

Thus the operating regime of the column can be assumed.

*Example 6. Time for merging of frozen cylinders or formation of frozen walls, with allowance for interaction of freezing columns*

Given:

$r$ = 0.106 m.

$\lambda_{th}$ = 2 kcal/m-hr-degree.

$C_{fr}$ = 480 kcal/m$^3$-degree.

$l$ = 1.5 m (spacing of columns).

$2l$ = 3 m is the thickness of the frozen wall.

The meanings of the symbols appear in Example 5.

To determine the time to merging of the frozen cylinders and formation of a frozen wall, we use the Fourier number of Eq. (29) (dimensionless time):

$$\text{Fo} = \lambda_t\tau/C_{fr}r_c^2 \text{ or } \tau = \text{Fo}C_{fr}r_c^2/\lambda_t.$$

For the value $h/r$ = 1.5/0.106 = 14.1, the nomogram of Fig. 22 gives Fo for the condition of merging of the frozen cylinders and formation of a frozen wall:

$$Fo=82;\ \tau=\frac{82\cdot 480\cdot 0{,}106^2}{2}=221\text{ hr}\approx 9{,}2\text{ days.}$$

$$Fo=170;\ \tau=\frac{170\cdot 480\cdot 0{,}106^2}{2}=4584\text{ hr}\approx 19{,}1\text{ days.}$$

*Comparison of the times needed for the frozen cylinders to merge, with and without allowance for interaction of the columns*

The time required for the merging of the ice-soil cylinders, without regard to the interaction of the freezing columns, comes from Eq. (26):

$$R_1 = \sqrt[3]{\frac{3\tau_1 t_c r_c \lambda_{fr}}{q}} + 0{,}5\, r_c,$$

where $q = 80{,}000 \cdot W_0 = 80{,}000 \cdot 0.2 \cdot 1.6 = 25{,}600$; $W_0 = W_t \gamma_{sk}$; $t_c = -31° + 4° = -27°C$; and $\lambda_{fr} = 2.4$ kcal/m-hr-degree.

Let $\tau_1 = 400$ hr be given. Then

$$R_1 = \sqrt[3]{\frac{3 \cdot 400 \cdot (-27)\, 0{,}106 \cdot 2{,}4}{25\,600}} + 0{,}5 \cdot 0{,}11 = \sqrt[3]{\frac{8250}{25600}} + 0{,}05 = 0{,}75 \text{ m}.$$

Thus, $\tau_1 = 400$ hr if interaction is not considered, but $\tau_1 = 221$ hr if interaction is taken into account.

Here $K_{spec} = 400/221 = 1.8$ is the coefficient of extension of the time to merging of the frozen cylinders without allowance for the interactions of the freezing columns.

*Example 7. Velocity of air in freezing columns and blower capacity*

The calculation is performed for the dam on the Irelyakh River.

Given:

$R_1$, the radius of the frozen cylinder after time $\tau_1$.

We determine the volume of the frozen cylinder around the freezing column by

$V_{cyl} = \pi(R_1^2 - r_c^2)\, h_{cm}$; $R_1 = 0.75$ m; $r_c = d_c/2 = 0.11$; and

$h_{cm} = 20$ m (mean column length).

Then

$$V_{cyl} = 3.14\,(0.75^2 - 0.11^2)\, 20 = 34.4 \text{ m}^3.$$

The crest length of the dam is $L = 320$ m. The spacing between column axes is $l = 1.5$ m. Then the number of columns will equal

$$n = L/1.5 = 320/1.5 = 213.$$

The volume of the frozen cutoff formed after time $\tau_1$ equals

$$V = V_{cyl} n = 34.4 \cdot 213 = 7350 \text{ m}^3.$$

The quantity of heat liberated when the soil in the whole cutoff volume freezes is given by the formula

$$q_{fr} = qV,$$

where $q$ is the value computed by the equation

$$q = 80{,}000W_0 = 80{,}000 \cdot 0.15 \cdot 1.7 = 20{,}400 \text{ kcal/m}^3.$$

Then $W_0 = W_t \gamma_{sk} = 0.15 \cdot 1.7$ and

$$q = 20{,}400 \cdot 7350 = 150{,}000{,}000 \text{ kcal}.$$

The cold transmission into the soil as the cold outdoor air moves through the annular space in the column is

$$q = 150{,}000{,}000 = \Phi \gamma_{air} C_{air} (t_{in} - t_{out}),$$

where $\Phi$ is the total volume of air passed through all the columns in the first freezing period; $\gamma_{air}$ is the specific gravity of air, kg/m$^3$ (1293 kg/m$^3$ at $t = 0$); $C_{air}$ is the specific heat of air, 0.24 kcal/m$^3$-degree; and $t_{in} - t_{out}$ is the difference between the inlet and outlet air temperatures, equal to $20 - 18 = 2°$.

In this example the somewhat low value $t_{in} - t_{out} = 2°$ was assumed. The most efficient freezing is achieved at $t_{in} - t_{out} = 8$ or 9°C.

If we specified the running time of all blowers as $\tau_1 = 1200$ hr = 50 days, then the total capacity would equal

$$P = \frac{\Phi}{\tau_1} = \frac{242\,000\,000}{1200} = 201\,000 \text{ m}^3/\text{hr}.$$

If all freezing columns are divided among seven sections ($m = 7$), then the capacity of each blower serving one of the $m$ sections of the freezing system will be

$$P' = \frac{P}{m} = \frac{201\,000}{7} = 28\,800 \text{ m}^3/\text{hr}.$$

If the area $\omega$ (in m$^2$) of the annular cross-section and the number $N$ of columns in a section served by one blower are known, the mean air velocity in the annular space is

$$V = \frac{P}{3600\,N\omega} = \frac{28800}{3600 \cdot 31 \cdot 0{,}0157} = 1{,}64 \text{ m/sec};$$

$$N = \frac{n}{m} = \frac{213}{7} = 30{,}4 = 31 \text{ columns};$$

$$\omega = \pi \left[\left(\frac{d_2}{2}\right)^2 - \left(\frac{d_1}{2}\right)^2\right] = 3{,}14\,(0{,}1^2 - 0{,}07^2) = 0{,}0157 \text{ m}^2;$$

where $d_2 = 0.2$ m is the inside diameter of the outer pipe and $d_1 = 0.14$ m is the diameter of the inner pipe.

On the basis of practical data, $V_{air}$ should not be less than 2 m/sec. Since the value just obtained is less than 2 m$^3$/sec, the back conversion gives the required blower capacity as

$$P_x = 3600 \cdot 31 \cdot 0{,}0157 = 3500 \text{ m}^3/\text{hr}.$$

Therefore, the operation of $n$ columns with $V$ = 2 m/sec requires a single-blower capacity of 3500 m$^3$/hr.

*Example 8. Approximate mean air velocity in the annular space for given blower capacity*

Let us determine the mean air velocity for a blower capacity given as $P$ = 3500 m$^3$/hr.

The mean air velocity will equal

$$V = \frac{0{,}35P}{d_2^2 - d_1^2},$$

$$V = \frac{0{,}35 \cdot 3500}{0{,}2^2 - 0{,}14^2} = 61\,250; \quad \frac{61\,250}{3600} = 17{,}01 > 2 \text{ m/sec}.$$

*Example 9. Mean temperature of frozen cutoff*

Let us use Eq. (42) to find the mean temperature over the volume of the frozen cutoff or of the dam on the Irelyakh River.

The mean temperature over the cutoff volume, for complete merging of the cylinders, will equal

$$t_m = t_{an}\left(0{,}32 + 0{,}8\frac{d}{l} - 0{,}2\frac{l}{s}\right) = -15\left(0{,}32 + 0{,}8 \times \right.$$
$$\left. \times \frac{0{,}22}{1{,}5} - 0{,}2\frac{1{,}5}{12}\right) = -6{,}2°\text{C},$$

where $t_{an}$ = 0.84$t_{vp}$ + 0.09$h$ = 0.84(-20) + 0.09·20 = -15°; $t_{vp}$ is the outdoor-air temperature averaged over the ventilation period, -20°C; $d$ is the outside diameter of the column, 0.22 m; $h$ is the height of the column, 20 m; $l$ is the spacing in the cutoff, 1.5 m; and $s$ is the minimum thickness of the cutoff in the plane in which the cylinders merge, 12 m (measured along the normal to the longitudinal axis of the cutoff).

Appendix 4.
Thermotechnical computer calculations
of the non-steady-state temperature regime
of a homogeneous non-filter earth dam
under severe climatic conditions

(1) The problem of the non-steady-state temperature field in a non-filter earth dam with allowance for the latent heat of phase transformations of soil moisture can be regarded as a two-dimensional Stefan problem. It is assumed that this temperature field is described by the thermal-conductivity equation in a region representing the outline of the dam with foundation (Fig. 33). That is, in the frozen zone, where $t < t_p$,

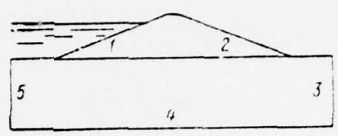



Fig. 33. Calculated outline of a dam and foundation.

$$C_{fr}\frac{\partial t}{\partial \tau}=\lambda_{fr}\left(\frac{\partial^2 t}{\partial x^2}+\frac{\partial^2 t}{\partial y^2}\right),$$

and in the thawed zone, where $t > t_p$,

$$C_{th}\frac{\partial t}{\partial \tau}=\lambda_{th}\left(\frac{\partial^2 t}{\partial x^2}+\frac{\partial^2 t}{\partial y^2}\right),$$

where $t$ is the soil temperature, °C (K); $C_{fr}$ and $\lambda_{fr}$ are respectively the volumetric specific heat (kcal/$m^3$-degree or kJ/$m^3$-K) and the thermal conductivity (kcal/hr-m-degree or W/hr-m-degree) of the frozen soil; $C_{th}$ and $\lambda_{th}$ are the corresponding quantities for the thawed soil; $\tau$ is the time, hr; and $t_p$ is the temperature at which phase transformations of the soil moisture occur (°C or K). The interface between thawed and frozen zones varies in time and is not known beforehand. On this boundary, which is sought in the solution of the problem, the Stefan condition is satisfied:

$$t=t_p;\ \lambda_{fr}\frac{\partial t}{\partial n}\Big|_{fr}-\lambda_{th}\frac{\partial t}{\partial n}\Big|_{th}=\rho\gamma W\frac{dl}{d\tau},$$

where $n$ is the normal to the interface; $\rho$ is the latent heat of the phase transformation per unit mass of water, kcal/kg; $\gamma$ is the specific

gravity of the soil, kg/m$^3$; $W_t$ is the total moisture content of the soil;* and $dl/d\tau$ is the rate of motion of the interface between the thawed and frozen zones, m/hr.

The temperature in the dam embankment and foundation can be given as an initial condition. In portion *1* of the upstream slope (see Fig. 33) and on the reservoir bottom, the boundary condition has the form

$$t = t_1, \tag{46}$$

and on surface *2*, which is adjacent to the air,

$$t = t_2, \tag{47}$$

where $t_2$ and $t_1$ are the month-by-month long-term mean temperatures of air** and water in the reservoir.

On sections *3* and *5* (Fig. 33) the boundary condition was given in the form $\partial t/\partial x = 0$, which reflects the absence of heat fluxes through these surfaces. On boundary *4* at depth $2h$ (where $h$ is the height of the dam), the boundary condition was taken in the form

$$t = t' = \text{const.}$$

The computer calculation of the temperature regime was carried out by an explicit six-point difference scheme based on the heat balance. This method was first tested in methodical calculations of one-dimensional and two-dimensional Stefan problems.

The results of calculations of the temperature regime for homogeneous dams with symmetrical cross-sections are presented below. The dams were assumed 30 m high, with a 25 m water level in the upper pool and a crest width of 4-5 m. Only the operating period was considered. It was assumed that this period begins in October, that the reservoir is filled by that time, and that the lower pool contains no water at any time in the period of the calculations.

The thermophysical characteristics of the soils in the embankment and foundation, and also the other quantities needed for the calculation, were taken with the following values.

Moisture content $W_t = 0.3$ in the embankment and foundation; specific gravity of soil $\gamma = 1800$ kg/m$^3$ in the embankment, $\gamma = 1600$ kg/m$^3$ in the foundation.

$$\lambda_{th} = 1.5 \text{ kcal/m-hr-degree} = 1.74 \text{ W/m-degree.}$$

---

*$W_t$ is defined as the ratio of mass of water per unit soil volume to the mass of the unit soil volume including its water.

**The warming effect of snow cover on the temperature regime of the dam was not taken into account in these calculations.

$\lambda_{fr}$ = 1.8 kcal/m-hr-degree = 2.088 W/m-degree.

$C_{th}$ = 690 kcal/m$^3$-degree = 2898 kJ/m$^3$-degree.

$C_{fr}$ = 480 kcal/m$^3$-degree = 2016 kJ/m$^3$-degree.

These data come from the Chapter "Foundations and footings on permafrosts" in the Construction Standards and Regulations and correspond to sandy loams at the indicated density and moisture content.

The latent heat of the phase transformations of a unit mass of water is $\rho$ = 80 kcal/kg (336 kJ/kg), and the temperature of the phase transformation is $t_p$ = -0.05°C (273.1 K).

Table 14 gives month-by-month long-term mean temperatures of water and air for the region around Mirnyy in the Yakut ASSR.

Table 14.

| Month | Temperature | | | |
|---|---|---|---|---|
| | air | | water | |
| | K | °C | K | °C |
| 1 | 238.15 | −35 | 274.4 | 1.25 |
| 2 | 240.65 | −32.5 | 274.4 | 1.25 |
| 3 | 255.75 | −17.4 | 273.66 | 0.51 |
| 4 | 267.95 | −5.2 | 273.9 | 0.75 |
| 5 | 276.05 | 2.9 | 278.25 | 5.1 |
| 6 | 282.95 | 9.8 | 286.25 | 13.1 |
| 7 | 289.15 | 16 | 290.45 | 17.3 |
| 8 | 287.75 | 14.6 | 286.45 | 13.3 |
| 9 | 277.35 | 4.2 | 282.25 | 9.1 |
| 10 | 267.35 | − 5.8 | 276.65 | 3.5 |
| 11 | 243.95 | −29.2 | 274.65 | 1.5 |
| 12 | 237.95 | −35.2 | 274.41 | 1.26 |

(2) For a preliminary estimate of the temperature regime of the dam and foundation, and for a determination of the initial and boundary conditions in the foundation, two one-dimensional problems were solved.

*(a) One-dimensional Stefan problem of the soil-temperature distribution over depth.* Condition (47) was taken at the soil surface adjacent to the air; the condition $\partial t/\partial y = g$ was assumed for a depth of 200 m, where $g$ is the geothermal gradient. The value of $g$ was arbitrarily assumed equal to 0.03 degree/m. This condition allows for the presence of a continuous heat flux from the interior of the earth (the $y$ axis points down). The initial condition is $t(y, 0)$ = -2°C over the whole depth, and the period of the calculations lasts 100 years.

After such a long time, the temperature distribution is nearly steady-state and has little dependence on the starting data. Figure 34 presents the results of the calculation for January, May and October of

the 100th year. Seasonal temperature variations do not penetrate deeper than 12-15 m.

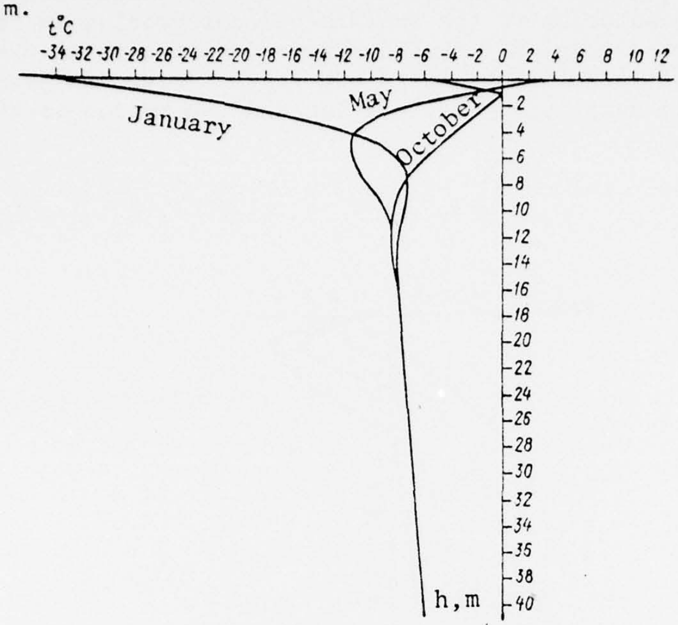



Fig. 34. Temperature distribution over soil depth at different times of the year, obtained by solution of the Stefan problem.

The results of a calculation of this problem were used in the two-dimensional problems to set the boundary condition at section *4* (Fig. 33) and the initial conditions in the dam foundation.

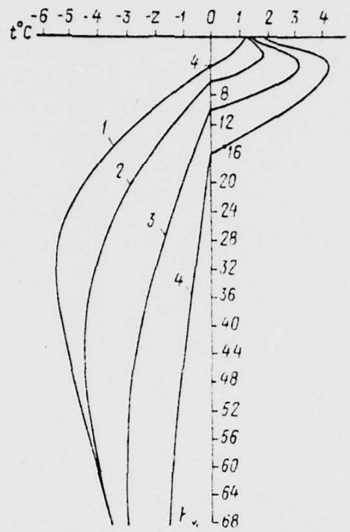
(b) *One-dimensional problem of the thawing of the reservoir bed after its filling.* In this problem, condition (46) is imposed at the soil surface of the reservoir bed. The initial condition is taken from the solution of the preceding problem. For the rest, the problem is analogous to the preceding one. Fig. 35 presents calculated results for the 5th, 10th, 25th and 75th years. Fig. 36 shows how the thawing depth of the reservoir bed depends on time. This relation can be described by the empirical formula $\xi = 2\sqrt{\tau}$, where $\xi$ is the thawing depth, m; and $\tau$ is the time, years.

The results of a calculation of this problem can serve to set the boundary condition on section *4* (Fig. 33), and also to correct the solution of the two-dimensional problems.

(3) Two-dimensional problems

(a) *Dam with 1 : 4 slopes, thawed variant.* Starting data: $t_0 = +1°C$

in the embankment; the initial temperature in the foundation is set on the basis of the solution to the one-dimensional problem in Paragraph 2 (a). On boundary *4* (Fig. 33) the boundary condition is $t' = -4.5°C$. The period of the calculations lasts 200 years. Fig. 38 shows the temperature field obtained by calculation and the motion of the freezing limit.




Fi[illegible]. Temperature distribution under the reservoir bottom after the reservoir has been filled, obtained by numerical solution of the Stefan problem. (1) 5 years; (2) 10 years; (3) 25 years; (4) 75 years.


Fig. 36. Thawing depth in the reservoir bed as a function of time, from numerical solution of the Stefan problem. Here $x$ is the thawing depth in the reservoir bed.

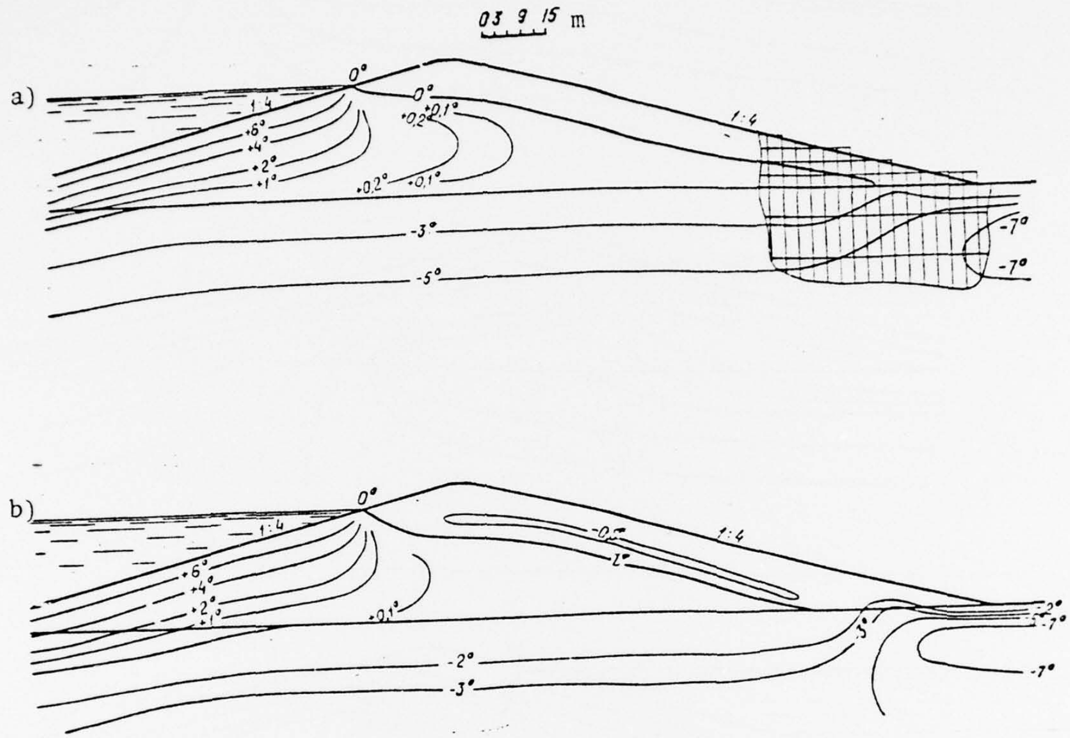



Fig. 37. Time variation of the temperature field in a thawed earth dam with 1 : 4 slopes. Initial boundary temperature is +1°C. This page: (a) 5 years; (b) 10 years. Parts (c), (d), (e) and (f) appear on the next page.

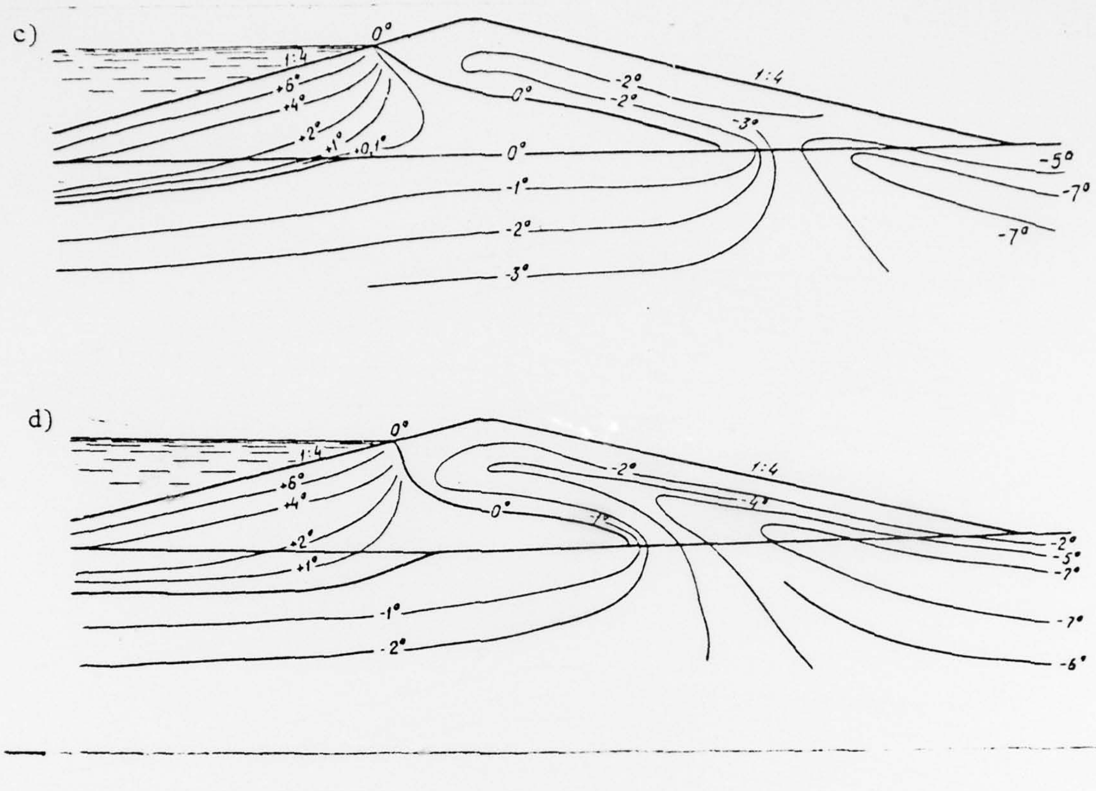



Fig. 37, continued. (c) 25 years; (d) 50 years; (e) 100 years; (f) 200 years.

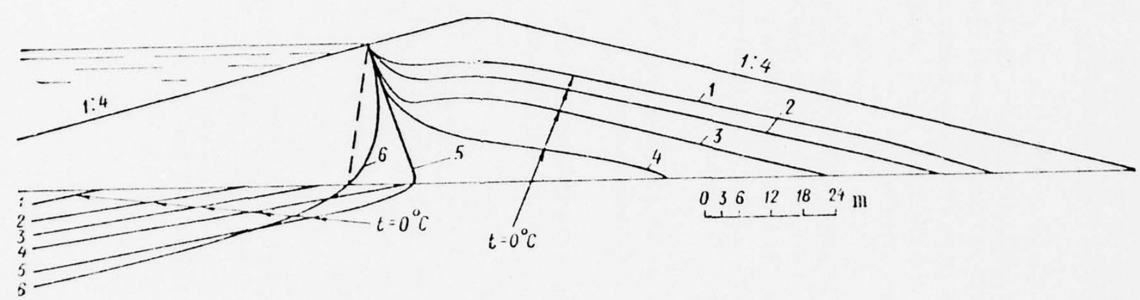



Fig. 38. Motion of the boundary of the phase transformation in a thawed earth dam with 1 : 4 slopes. Initial boundary temperature is +1°C. (1) 5 years; (2) 10 years; (3) 25 years; (4) 50 years; (5) 100 years; (6) 200 years.

The freezing front moves from the surface of the downstream slope into the embankment of the dam. Over the first 50 years, the front moves parallel to the surface of the downstream slope. The freezing depth in this period is proportional to $\sqrt{\tau}$. After this period, the freezing boundary gradually moves toward the upstream slope, approaching its limiting (steady-state) position. The location of the freezing boundary, according to I. S. Moiseyev's solution of the steady-state problem with yearly-average water and air temperatures, appears as a dashed line in Fig. 38.

By the end of the calculated period, a thick frozen layer has formed in the downstream slope. After the 100-year period of the calculations, the temperature field is nearly one-dimensional down to a depth of 15-20 m along a normal to the surface of the downstream slope and at a sufficient distance from the crest.

Through the year, phase transformations occur down to the active-layer depth on the surfaces of the downstream slope and the crest. The reservoir bed thaws to a depth proportional to $\sqrt{\tau}$. In 100 years, the bed thaws by approximately 20 m. According to the calculation, thawing continues even after the 100 years.

The phase transformation also takes place along the joint between the thawed dam and the frozen foundation.

The time required for the temperature field in the given dam to enter a quasi-steady state is, according to the calculation, on the order of 100 years.

(b) *Dam with 1 : 4 slopes, frozen variant.* Starting data: $t_0$ = -1°C in the embankment; other data as in the preceding problem. The period of the calculation lasts 100 years. The temperature fields obtained by calculation appear in Fig. 39. Figure 40 shows how the thawing boundary moves.

In this problem, in distinction to the preceding one, the dam embankment thaws from the upstream face. The thawing boundary moves into the embankment, remaining mostly parallel to the upstream slope. Over the first 50-100 years, according to the calculation, the thawing depth is proportional to $\sqrt{\tau}$. By the end of 100 or 200 years, the thawing boundary is stabilized in the embankment.

From the downstream side, freezing goes far faster and more intensively than for the thawed variant in the preceding problem. In the frozen section, the temperature field is set in the first 10 years, and changes little afterward.

By the end of the calculated period, the temperature field approximates that formed in the preceding problem. The temperature field does not depend on the starting conditions after a great length of time (on the order of 100 years).

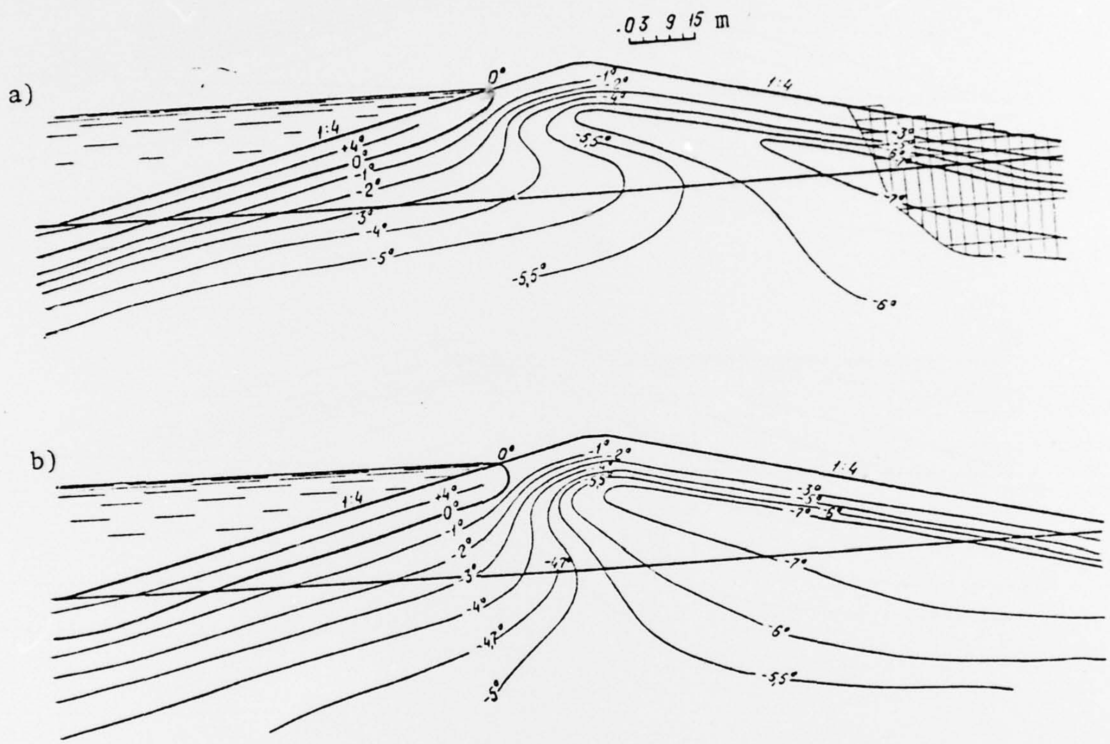



Fig. 39. Time variation of the temperature field in a frozen earth dam with 1 : 4 slopes. Initial boundary temperature is -1°C. This page: (a) 5 years; (b) 10 years. Parts (c), (d) and (e) appear on the next page.

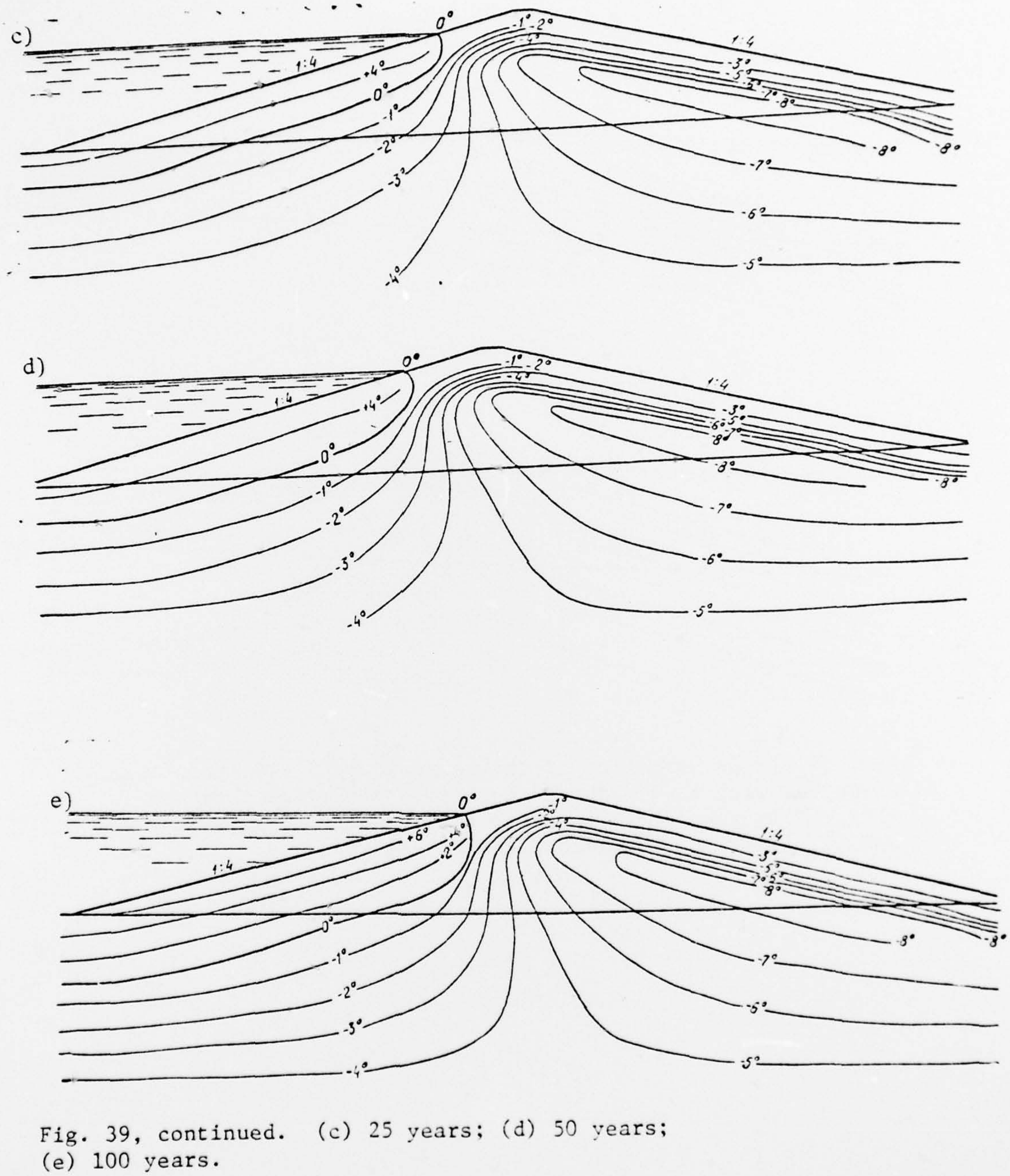



Fig. 39, continued. (c) 25 years; (d) 50 years; (e) 100 years.

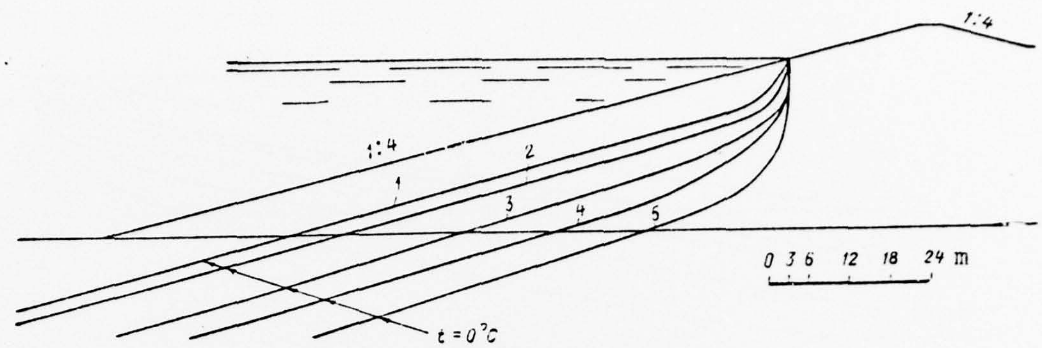



Fig. 40. Motion of the boundary of the phase transformation in a frozen earth dam with 1 : 4 slopes. Initial boundary temperature is -1°C. (1) 5 years; (2) 10 years; (3) 25 years; (4) 50 years; (5) 100 years.

(c) *Dam with 1 : 6 slopes, thawed variant.* Initial and boundary conditions the same as in problem 3 (a). Calculated period, 100 years. Fig. 41 shows how the zero-degree (Celsius) isotherm moves.

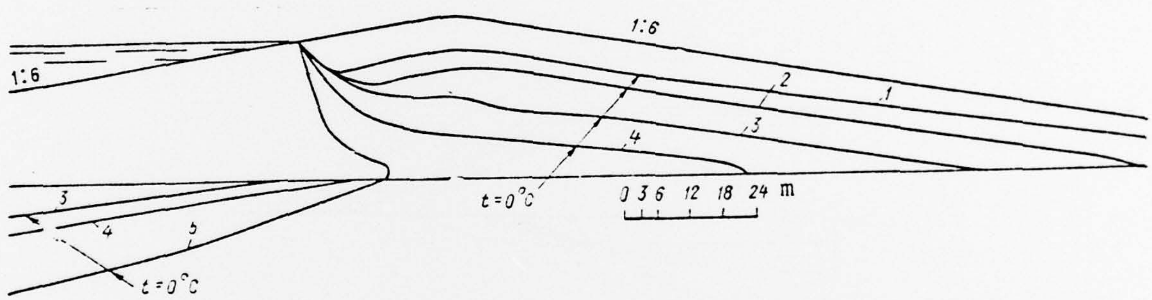



Fig. 41. Motion of the boundary of the phase transformation in a thawed earth dam with 1 : 6 slopes. Initial boundary temperature is +1°C. (1) 5 years; (2) 10 years; (3) 25 years; (4) 50 years; (5) 100 years.

The calculated result as a whole is analogous to that in Paragraph 3 (a).

(d) *Dam with 1 : 6 slopes, frozen variant.* Initial and boundary conditions as in problem 3 (b). Figure 42 shows the motion of the zero-degree (Celsius) isotherm.

The calculated results are analogous to those of Paragraph 3 (b).

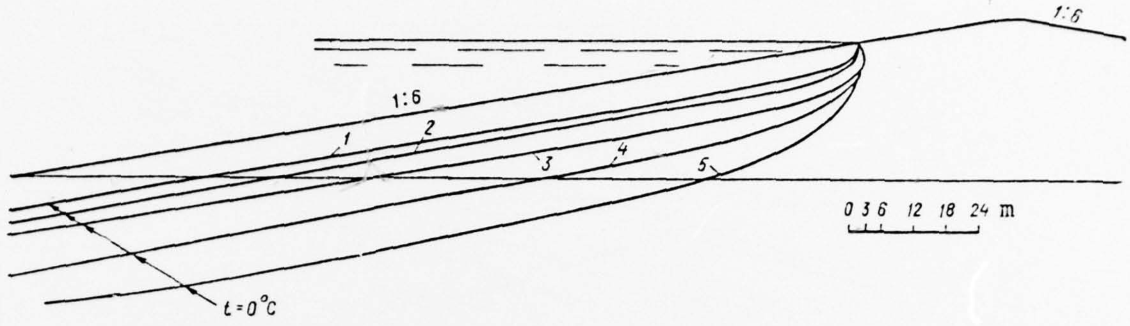



Fig. 42. Motion of the boundary of the phase transformation in a frozen earth dam with 1 : 6 slopes. Initial boundary temperature is -1°C. (1) 5 years; (2) 10 years; (3) 25 years; (4) 50 years; (5) 100 years.

(e) *Dam with 1 : 2 slopes, thawed variant.* Initial and boundary conditions nearly the same as in problem 3 (a). The calculated period lasts 100 years. Figure 43 shows how the zero-degree (Celsius) isotherm moves in the embankment of the dam.

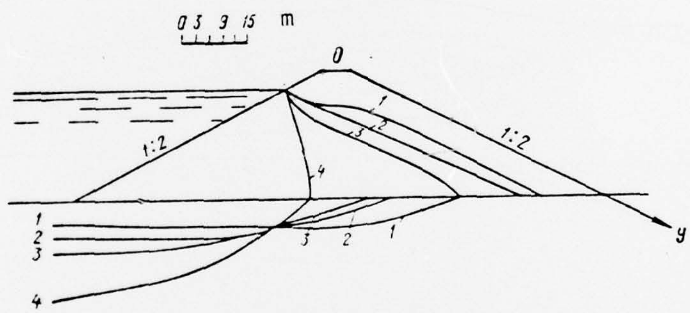



Fig. 43. Motion of the boundary of the phase transformation in a thawed earth dam with 1 : 2 slopes. Initial boundary temperature is +1°C. (1) 5 years; (2) 10 years; (3) 25 years; (4) 100 years.

On the basis of the calculated results presented here, it can be concluded that a pre-frozen dam is considerably more expedient, since it can take up pressure from water in the reservoir immediately after it is built. The conditions of thawed and frozen variants will be alike only under steady-state conditions; in practice, according to the calculations, this means not until 50-100 years after they are constructed.

Figures 44 and 45 present the time variation of $K_0$ calculated for homogeneous dams with 1 : 4 and 1 : 6 slopes. $K_0$ is the ratio of the

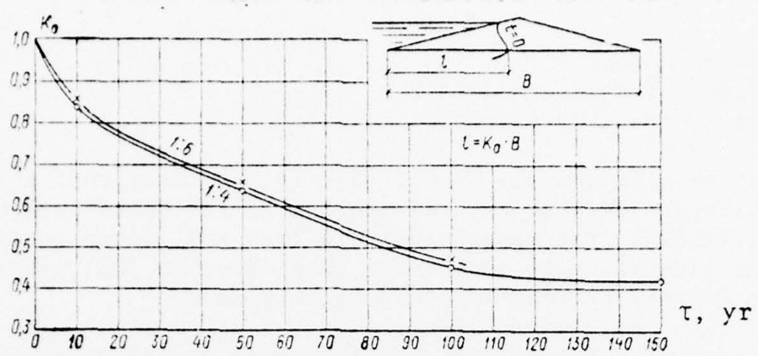



Fig. 44. $K_0$ for thawed dams. According to I. S. Moiseyev's solutions for steady-state conditions, $K_0$ = 0.398 for 1 : 4 slopes.

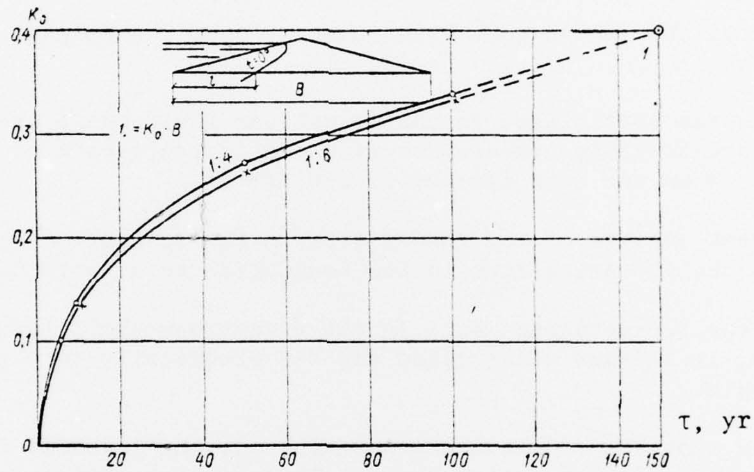



Fig. 45. $K_0$ for frozen dams. According to I. S. Moiseyev's solutions for steady-state conditions, $K_0$ = 0.398 for 1 : 4 slopes.

width of the thawed zone at the dam foundation $l$ to the total width of the dam $B$.

The distance $l$ from the upstream slope to the boundary between thawed and frozen zones, measured along the foundation, is given by

$$l = K_0 B.$$

Appendix 5.
Approximate evaluation
of the strength of a frozen cutoff
in the initial period of operation

The planning of a frozen dam should take into account that the freezing of the downstream shell by natural cooling of the downstream-slope surface and lateral transmission of cold from the frozen cutoff may continue for several years (not fewer than three to four years) if the soil is not frozen layer-by-layer as it is placed.

The frozen core of a frozen dam should be wide enough to make the structure stable when the reservoir is filled before the downstream shell has completely frozen.

The overall stability of an earth dam with frozen core should be evaluated by calculations using an approximate plane-shear scheme (Fig. 46), under the following simplifications:

Width of the frozen core $b$ is constant over the height of the dam.

The frozen soil placed in the downstream shell still preserves an above-freezing temperature in a considerable part of its volume when the core freezes to a width $b$.

The heat content of the thawed zone in the downstream shell thaws the icy permafrost in its foundation to a depth $h_{thd}$.

When the ice-saturated soil in the downstream-shell foundation thaws, it becomes waterlogged and has practically zero shear strength.

By the time the core thickness reaches $b$, the icy soil in the upstream-shell foundation thaws. This happens either under the action of reservoir heat or when a talik develops in the construction period.

The approximate calculation takes the angle of internal friction

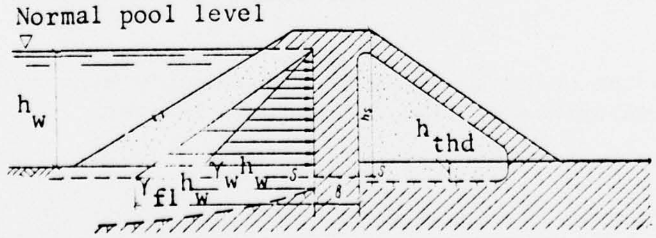



Fig. 46. Computational scheme for determining the strength of a frozen cutoff.

and the cohesion of the thawing foundation soil equal to zero, because these values are small.

When the upstream shell is dumped from thawed compacted soils, the lateral pressure of the soil on the vertical surface of the frozen core can be assumed equal to zero, since the thaw settlement of the foundation is accompanied by the appearance of tensile stresses and settlement cracks where the upstream-shell soil makes contact with the pressure face of the core.

Above normal pool elevation, the core experiences no lateral pressure from the frozen soil layer on the crest.

If the upstream shell is erected from frozen soils that will be structurally unstable on thawing, the calculation should allow for lateral pressure exerted by the waterlogged soil, which has zero angle of internal friction and zero cohesion.

The core may shear in some plane $s$-$s$ under the action of the hydrostatic pressure of water and the lateral pressure of the fluidized soil of the upstream shell.

The plane $s$-$s$ lies at a critical depth $h_s$, for which the quantities $\phi_{tg}$ and $C_{tg}$ of the thawing soil can be assumed equal to zero.

The downstream shell has no cohesion with the foundation, because the values of $\phi_{tg}$ and $C_{tg}$ are zero in the layer where the thawing soil of the base makes contact with the foundation. Thus the downstream shell does not take part in resisting shearing forces (lateral pressure from soil in the upstream shell, hydrostatic pressure of water) until it has completely frozen through.

The shear strength of the frozen soil in the core $R^n_{sh}$ is taken from Table 7 for the mean design temperature of the frozen wall at depth $h_s$.

The shear strength of the frozen core in plane $s$-$s$ for the period when the downstream shell has zero shear strength can be approximately estimated from the following formulas:

(a) For the case of no lateral pressure from soil in the upstream shell,

$$b \geq \frac{\gamma_w h_w^2}{2(\gamma_{fr} h_s + R^n_{sh})} . \tag{48}$$

(b) For the case of simultaneous application of hydrostatic pressure and lateral pressure from the thawed soil,

$$b \geq \frac{h_w^2(\gamma_{fl} + \gamma_w)}{2(\gamma_{fr}h_s + R^n_{sh})}. \tag{49}$$

In Eqs. (48) and (49), $\gamma_{fr}$ is the specific gravity of frozen soil in the core, kg/m$^3$; $\gamma_w$ is the specific gravity of water, kg/m$^3$; $h_w$ is the depth of water saturating the upstream slope, m; $\gamma_{fl}$ is the specific gravity of the fluidized soil of the upstream shell; and $h_s$ is the height of the frozen core above the calculated shear plane $s$-$s$.

Two sample calculations aimed at determining the minimum thickness follow.

*Example*

To determine the minimum width of the frozen core accepting a water head after the merging of frozen cylinders but before the freezing of the downstream shell of the dam.

In the calculation, the temperature of the soil in the downstream shell of the dam is assumed above zero, and the soils in the assumed shear plane are taken to have zero strength (see Fig. 46).

Given:

$h_s$ = 30 m.

$h_w$, height of that part of the core taking up water and soil pressure from the upper-pool side, 25 m.

$\gamma_{fl}$, specific gravity of fluidized soil in the upstream shell, 1.1 ton/m$^3$.

$\gamma_{fr}$, specific gravity of frozen soil in the dam core, 1.84 ton/m$^3$.

$\gamma_w$, specific gravity of water, 1 ton/m$^3$.

$R^n_{sh}$, shear strength of the frozen soil, 25 ton/m$^2$.

If there is no lateral pressure from the soil in the upstream shell,

$$b \geq \frac{\gamma_w h_w^2}{2(\gamma_{fr}h_s + R^n_{sh})} = \frac{1 \cdot 2.5^2}{2(1.84 \cdot 30 + 25)} = 3.9 \text{ m.}$$

If hydrostatic pressure and lateral pressure from the thawed soil are simultaneously applied,

$$b \geq \frac{h_w^2(\gamma_{fl} + \gamma_w)}{2(\gamma_{fr}h_s + R^n_{sh})} = \frac{2.5^2(1.1 + 1)}{2(1.84 \cdot 30 + 25)} = 8.16 \text{ m.}$$

Appendix 6.
Combating the destruction
of icy reservoir banks

In the construction of reservoirs in river valleys made up of loose ice-saturated rocks with ground-ice inclusions and features of natural thawing of ground ice, it is necessary to take a series of steps to prevent further thawing of ground ice and destruction of the banks in regions where the dam cuts into the valley walls and near existing structures.

Denuded ice wedges and other ground-ice bodies thaw faster than does the soil enclosing them. Unlike soil, an ice body thawing under water acts as neither heat shielding nor light screening over the frozen soils of the bed. Under the severe conditions of the arctic tundra, small bodies of water accumulate more heat in the course of the summer than they dissipate in winter. Starting from a depth of 1.4-1.6 m, the summer thawing of the bottom exceeds the winter freezing. The total heating of the frozen ground is increased, and the progressive development of a talik, with thawing and erosion of the icy banks, begins.

As the reservoir water level rises, reworking of the banks speeds up sharply, because wave action is intensified. Waves continually wash away the thawed soil, denuding ice bodies in the bank bluff. The bank thaws, and deep niches (up to 7-10 m) are cut at the water level. Caved blocks of frozen soil, reaching lengths of 0.5-0.7 km, are completely eroded over one to two months. A high ice-saturated bank, which is most prone to thermal abrasion, retreats at up to 10 m/yr (in isolated cases this value can reach 15-20 m/yr around the whole perimeter).

There are two methods for setting up a heat-shielding layer to hold up the reworking of banks and the thawing of large ice inclusions in dam abutments:

(a) A layer of weakly-pervious soil is dumped and compacted and provided with erosion protection. This layer simultaneously acts as an impervious lining.

(b) The reservoir is temporarily (1-2 yr) operated at water levels higher than the normal pool elevation. After the ice has thawed, the original shoreline has been reworked, and the soils around the reservoir perimeter have been compacted, a trail of thawed soils forms. These resist water action and protect the icy bank from further thawing and damage.

A heat-shielding layer artificially dumped or formed by natural thawing can be given wave protection by a dumped layer of gritty soil with a rockfill cover. The destructive action of ice fields should also be taken into account.

The protection of ice-saturated banks with slabs or rockfill is

ineffective unless a heat-shielding soil layer is provided. Such a heat shield can be made from low-moisture-content thawed soils found in the beds of young, shallow thermokarst lakes. The soils should be dried before use, with the help of elementary drainage ditches. Other soils containing a sufficient amount (not less than 20%) of soil separates (particles with $d$ < 2 mm) can also be used.

Provided that the thawed soil collapses sufficiently fast and has little effect on heat exchange between the wall of frozen soil and the water, the retreat of the underwater ice-saturated bench in the bank slope can be determined by the formula

$$L = \frac{\alpha t_1 \tau}{\rho W \cdot 0{,}9}, \tag{50}$$

where $L$ is the retreat of the bank due to thawing and thermal abrasion, from the moment when thaw settlement of the reservoir bed finishes, with complete thawing of the ground ice under the bed, m; $\rho$ is the latent heat of the phase transformation of soil moisture (80,000 kcal/m$^3$ ≈ 320 MJ/m$^3$); $W$ is the ice content (by volume) of the frozen soils in the bank slope, including buried ice, a fraction; $\alpha$ is the heat-transmission coefficient for free convection of water at the vertical ice-soil wall insulated from above, kcal/m$^2$-hr-degree; $\tau$ is the time, hr; and $t_1$ is the mean water temperature, °C.

Equation (50) does not allow for mechanical (wave) damage to the thawing bank.

The possibility of unconstrained retreat of an ice-saturated bank due to thermal abrasion (Fig. 47) is determined by the formula

$$H_{des} < h - \eta h, \tag{51}$$

where $H_{des}$ is the height of the icy bank above the water level; $h$ is the thickness of the soil layer enclosing the ice; and $\eta$ is the relative height of this layer after thawing of the ice.

If thermokarst lakes exist in the region of dam construction, the possibility of stabilizing a high-ice-content bank when the reservoir is formed (Fig. 48) is determined by the formula

$$H_{stab} \geqslant h - \eta h; \tag{52}$$

$$\eta = \frac{h - (H + h_0)}{h}, \tag{53}$$

where $h_0$ is the depth of the analog thermokarst lake; and $H$ is the amount by which the surface of the ice-enclosing soil stands above the water in this lake.

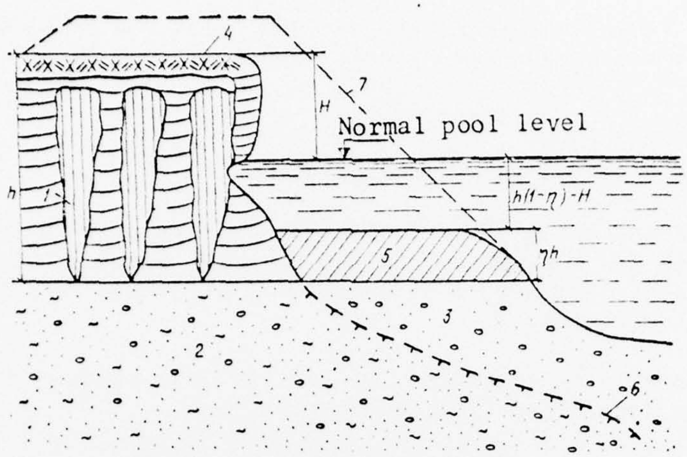



Fig. 47. Continuous destruction of a bank with inclusions of thick buried ice bodies. (1) Heavily-settling soils of finest clays, with inclusions of pure ice; (2) low-ice-content permafrosts; (3) thawed, low-settlement soils; (4) soils of active layer; (5) deposits of soil after thawing of ice inclusions; (6) permafrost table; (7) approximate outline of embankment needed to protect slope from destruction.

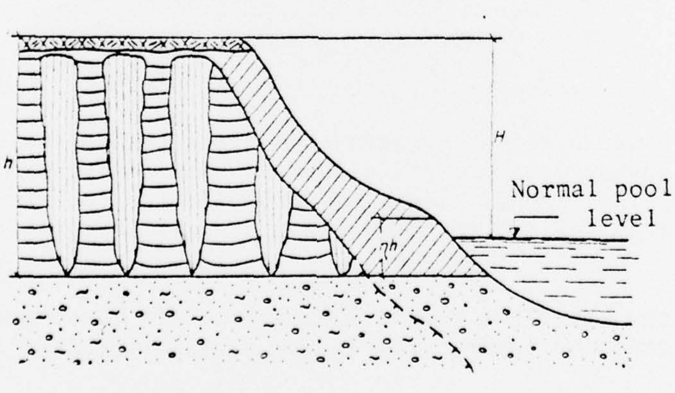



Fig. 48. Formation of a stable reservoir bank in regions where thick buried ice bodies develop. Identifications the same as in Fig. 47.

An ice-enclosing bank retreating without constraint can be protected from thermal abrasion by a layer of soil with a minimum height of

$$h_{pr} = h - H - \eta h, \tag{54}$$

which is shown as a dashed curve in Fig. 47.

This layer can be protected from erosion due to changes in water level and wave action in the reservoir. Filter material and a protective layer of coarse-fragmented soil should be used, with allowance for the possible impact of ice fields.

If the bank is damaged by the scheme of Fig. 49, the maximum retreat of the bank $L$ can be determined from the formula

$$L = \frac{h - H_0 - \eta h}{\tan \psi}, \tag{55}$$

where $H_0$ is the initial height of the icy bank above normal pool elevation, m; and $\psi$ is the slope angle of the foot of the soil layer enclosing buried ice.

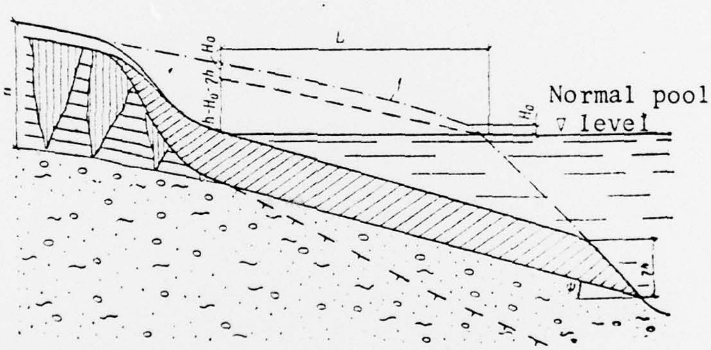



Fig. 49. Diagram for calculating the distance $L$ by which the waterline advances before the reservoir bank achieves a stable position at the toe of a layer of soil with ice inclusions inclined at an angle $\psi$ to the horizontal. (1) Original shape of bank.

The advance of the waterline toward such a bank can be stopped by the dumping of a protective layer, even before equilibrium is reached.

Appendix 7.
Combating frost heaving
and frost splitting

In the planning of dams for Far North regions, the following dangerous manifestations of frost heaving should be taken into account:

(a) Heaving of cohesive soils upon freezing, in the course of layer-by-layer winter placement.

(b) Heaving in the seasonally thawing layer at the crest and on the slopes of the dam, upon repeated freezing and thawing in the operating period (without heat-shielding layer or with a layer that is not thick enough).

(c) Heaving of foundations during dam construction and operation (at the toe and abutments of the dam).

When a thawed cohesive soil is frozen through once during winter placement in a dam embankment, the harmful effect of heaving can be effectively reduced by overcompaction of the soil beyond the maximum standard density. In repeated freezing and thawing, heaving results in the gradual decompaction and waterlogging of the soil, which cuts its strength and water-confining properties. Some reduction in heaving is achieved when the initial moisture content is decreased to values near the plastic limit $W_p$. The presence of coarse-fragmented inclusions in the quantity of 50% by weight cuts heaving by a factor of 1.5-2, although it does not permanently stabilize the soil against heaving. A sufficiently effective way of reducing heaving is an overload, up to 2.5-3 m thick, from non-heaving soil (Recommendations, Paragraph 5.46). In thawed dams, heaving in the seasonal freeze-thaw zone can be prevented by salting of the soil, to two-thirds of the depth of this layer, with NaCl, $CaCl_2$ and KCl. Salting to stop heaving is a rather long-term measure; its action extends to not fewer than 40-50 freeze-thaw cycles.

Frost clefts form as a result of deep winter cooling of the frozen-soil surface. Almost-rectangular networks form, with spacings of 5-25 m between cracks forming opposing sides. The blocks bounded by the cracks are usually called "polygons." The depth of a frost cleft may be from a few tens of centimeters to several meters. The possibility that a frost cleft will appear can be predicted from a complex of characteristics of the top 1.5-meter layer of soil. Frost clefts can appear in fine-grained sandy and clayey soils at a certain moisture content. If the total moisture content (ice plus unfrozen water) in the surface layer of the massif is less than three times the moisture content of any one of the 0.2-meter layers within the top 1.5-meter layer, then frost clefts can form only from the surface. If the moisture content of the top layer is greater than three times the moisture content of any one of the inner layers of the 1.5-meter soil layer, then frost clefts can also form from inside the layer. The placement of a heat-shielding layer 1.5 m or more in thickness, from soil with a large content of coarse particles (more than 70% $d \geq 2$ mm), but not containing silt

($d \leq 0.05$ mm) or a great quantity of large voids, makes it practically impossible for frost clefts to form. Following the recommendations on protection of dams against heaving automatically protects them against frost splitting too.